AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWENTY-SECOND ANNUAL CONVENTION

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J-3.—TECHNICAL PROBLEMS IN CEMENT MANUFACTURE.

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BY-LAWS.

AMERICAN CONCRETE INSTITUTE.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at the time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

SEC. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall elect by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-President and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members

on each special committee shall be appointed by the President, in consultation with the Chairman.

Sec. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

Sec. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President on the first regular session of the annual convention.

Sec. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

Article III.

MEETINGS.

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

DUES.

Section 1. The fiscal year shall commence July 1st.

Sec. 2. The annual dues shall be ten dollars ($10.00) payable annually in advance from first of the month following notification of the applicant of his election by the Board of Direction.

Sec. 3. Each member shall be entitled to receive one copy of one volume of the Proceedings for each membership year and additional volumes at a price fixed by the Board of Direction.

Sec. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

Article V.

STANDARDS.

Section 1. Proposed new or revised Standard Specifications, Standard Practice and Standard Definitions, when approved by a majority voting in the committee in which they originate, shall be submitted in the form adopted in the Standard Form of Standards to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF PROCEEDINGS OF THE TWENTY-SECOND ANNUAL CONVENTION.

Sherman Hotel, Chicago, Ill.

First Session, Tuesday, February 23, 1926, 2 P. M.

The convention was called to order by A. E. Lindau, president of the American Concrete Institute, who read a paper entitled

"The Trend of Institute Work."

The following papers were read and discussed:

"Correlated Considerations in the Design and Construction of Concrete Bridges," by A. Burton Cohen.

"Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders," by Harold M. Westergaard.

"Better Railway Track Support," by Frank H. Alfred (read in the author's absence by Mr. Grandy).

The last paper was followed by a discussion on "The Railroads and Concrete."

Second Session, Tuesday, February 23, 1926, 8 P. M.

A. E. Lindau in the chair.

The following papers were read and discussed:

"Earthquakeproof Construction," by H. M. Hadley.

"Earthquakes and Their Effect on Buildings," by Arthur L. Day.

The rest of the session was devoted to considerations of the "Question Box."

Third Session, Wednesday, February 24, 1926, 9.30 A. M.

Past-president W. K. Hatt in the chair.

The following papers were read and discussed:

"The Wilson Dam—How It Was Built," by Maj. M. C. Tyler.

"Controlling Quality of Concrete on the Wilson Dam," by John W. Hall.

"Seven Years' Experience With Job Control of the Quality of Concrete," by R. B. Young.

FOURTH SESSION, WEDNESDAY, FEBRUARY 24, 1926, 2 P. M.

M. M. Upson, vice-president, in the chair.

The following papers were read and discussed:

"Control of Concrete Mixtures on University of Pittsburgh Stadium," by W. S. Hindman.

Consideration was then given to the remaining part of the "Question Box" left over from the second session.

The following papers were read and discussed:

"New Experiences in Field Control," by John G. Ahlers.

"Application of the Water-Ratio Specification for Concrete," by F. R. McMillan and Stanton Walker.

The report of Committee E-5 on Aggregates was presented by its chairman, R. W. Crum. The report was adopted and the "Purchasing Specification for Aggregates" that it contains was accepted as a tentative standard for one year. Specification is not printed in this volume, but appears as a separate.

The report of Committee C-5 on Measurement of and Estimating Concrete was presented by A. B. Cohen, member of the committee.

Tentative Standard C5A-25T on Measurement of Concrete was referred to letter ballot for adoption as standard. (This standard was adopted by vote of the membership May 27, 1926, and appears in this volume of the Proceedings as C5A-26.)

FIFTH SESSION, WEDNESDAY, FEBRUARY 24, 1926, 8 P. M.

W. E. Hart in the chair.

The report of Committee E-6 on Destructive Agents and Protective Treatment was presented by its chairman, M. M. Upson, and secretary, F. R. McMillan.

The following papers were read and discussed:

"Concrete for Structure and Finish of Important Buildings," by David C. Allison. (Read by H. C. Boyden.)

"Architectural Concrete," by John J. Earley.

"Stucco Textures and Colors," by O. A. Malone.

The last paper consisted of a demonstration by Mr. Malone of the actual placing of various stucco finishes.

Sixth Session, Thursday, February 25, 1926, 9.30 A. M.

Richard L. Humphrey, past-president, in the chair.

The following papers were read and discussed:

"Uniformity of Building Code Regulation of Concrete Masonry Units," by Frank Cartwright.

"Building Regulations and Concrete Masonry Units," by George A. Hauser.

The report of Committee P-4 on Concrete Staves was presented by its chairman, W. O. Brassert. With minor corrections, approved by unanimous consent of the floor, Standard Specification on Concrete Staves, P4A-25T, was voted to be sent to letter ballot of the membership for approval as standard. (This standard was approved by the membership of the society on May 27, 1926, and appears in this volume of the Proceedings as Standard P4A-26.)

The report of Committee P-1 on Standard Concrete Building Units was presented by Chairman E. W. Dienhart. The report consists of amendments to Standard P1A-25 and Standard P1B-25. These were adopted by the meeting as tentative and are printed as a separate.

The following paper was read and discussed:

"Effect of Lime on Concrete Products," by Paul C. Cunnick.

P. H. Bates, chairman of Committee T-1 on Crazing, presented an oral progress report.

John J. Earley, chairman of Committee P-2 on Cast Stone and Architectural Concrete, presented a paper, "Future Work of Committee P-2."

The report of Committee P-6 on Concrete Products Plant Operation was presented by Benjamin Wilk, chairman. The report consisted of a paper on "How to Cure Concrete Building Units."

At a luncheon meeting details of the Wacker Drive, visited in the afternoon of February 25, were presented by the following speakers: T. A. Evans, engineer-in-charge of design, Board of Local Improvements, Chicago; Frank Herlihy, president, Mid-Continent Construction Co.; A. A. Levison, Blaw-Knox Co.; John J. Sloane, president, Chicago Board of Local Improvements.

The afternoon was devoted to the inspection of the Wacker Drive.

Seventh Session, Thursday, February 25, 1926, 8 P. M.

President A. E. Lindau in the chair.

The tellers announced the following elections:

President, M. M. Upson, of New York.
Vice-President (2-year term), John J. Earley, of Washington.

Secretary and Treasurer, Harvey Whipple, Detroit.
Directors, 3rd District, A. C. Tozzer.
2nd District, J. G. Ahlers, New York.
6th District, Rudolph J. Wig, Los Angeles.

(On account of the elevation of Mr. Earley from directorship in the 4th District to vice-president, a vacancy occurred in the 4th District directorship. The Board of Direction in accordance with the constitution filled the vacancy. P. H. Bates, Washington, D. C., is the new director.)

The following amendment to the by-laws legally presented to the meeting was adopted to be sent to letter ballot. (It was adopted by vote of May 27, 1926.) The amendment follows:

ARTICLE V.

STANDARDS.

SECTION 1. Proposed new or revised Standard Specifications, Standard Practice, and Standard Definitions when approved by a majority voting in the committee in which they originate, shall be submitted, in the form adopted in the Standard Form of Standards, to the secretary of the Institute 60 days prior to the opening of the annual convention at which they are to be presented. The secretary of the Institute shall cause these proposed new standards or revised standards to be printed as Proposed Tentative Standards and mailed to the full membership of the Institute thirty days prior to the opening of the convention. As there amended and approved, they shall be published in the Annual Proceedings, next issued as Tentative Standards. At a subsequent annual convention, they may again be offered unamended, by their originating committees as proposed standards, and as there approved by a majority of those voting, they shall be submitted to letter ballot of the Institute membership, to be canvassed within ninety days thereafter. Such proposed standards shall be considered adopted unless at least 10 per cent of those voting shall vote in the negative.

The change in the amendment lies entirely in the last sentence, which in the old by-laws read as follows:

> "Such proposed Standard shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative."

The Wason Medal was presented by President Lindau and C. E. Nichols to

E. A. Dockstader

for the most meritorious paper presented at the 1925 convention:

> "Report of Tests Made to Determine the Temperatures in Reinforced-Concrete Chimney Shells."

The secretary presented the so-called "Pen-Wiper" Trophy to Duff A. Abrams for leadership in sponsoring new Institute members, January 1, 1925, to February 15, 1926. The trophy consisted of an oversize pen-wiper and a set of desk fountain pens. The second prize, a fountain pen, was awarded to F. R. McMillan.

The following papers were read and discussed:

"Architectural Floors—Especially Terrazzo," by H. S. Wright.

The report of Committee J-2 on the American Concrete Institute Representation on Concrete Culvert Standards was presented to B. S. Pease, chairman of the Joint Culvert Pipe Committee. The report was accepted as a progress report.

The following papers were read and discussed:

"Efficiency in the Supervision of the Construction of Concrete Road Surfacing," by J. L. Harrison.

"Transverse Testing of Concrete," by H. F. Clemmer and Fred Burggraf.

EIGHTH SESSION, FRIDAY, FEBRUARY 26, 1926, 9.30 A. M.

President A. E. Lindau in the chair.

The report of Committee S-1 on Reinforced-Concrete Chimneys was read by its chairman, C. E. Nichols.

The following papers were read and discussed:

"Concrete Pavement Design," by L. W. Teller and J. T. Pauls.

"Extensibility of Concrete," by W. K. Hatt.

"Reinforced-Concrete Chimney Tests," by Benjamin Wilk.

"Relation of 7- and 28-Day Compressive Strengths of Mortar and Concrete," by Willis A. Slater. (Read by Stanton Walker.)

"Soaps as Integral Waterproofing for Concrete," by Alfred H. White and John H. Bateman.

"The Most Significant Tests for Concrete," by A. T. Goldbeck.

The report of Committee E-3 on Research was read by its chairman, H. F. Gonnerman.

The following paper was read and discussed:

"The Effect of Varied Curing Conditions Upon the Compressive Strength of Mortars and Concretes," by Herbert J. Gilkey.

THE WASON MEDAL.

Awarded Each Year to the Author of the Most Meritorious Paper Presented to the Previous Annual Convention.

Awarded 1926 to

E. A. Dockstader, for paper, "Report of Tests Made to Determine Temperatures in Reinforced-Concrete Chimney Shells, presented to the 1925 Convention.

Previous Awards.

1916 Convention Paper—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917 Convention Paper—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918 Convention Paper—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Convention Paper—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Convention Paper—W. A. Hull, "Fire Tests of Concrete Columns."

1921 Convention Paper—H. M. Westergaard, "Moments and Stresses in Slabs."

1922 Convention Paper—George E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

1923 Convention Paper—J. J. Earley, "Building the Fountain of Time."

1924 Richard L. Humphrey, for two papers, "Twenty Years of Concrete" and "The Promise of Future Development," presented to the 1924 Convention.

The Trend of Institute Work.

By A. E. Lindau.*

A generation ago Speaker Reed called this country a "Billion Dollar Country." Since that time billions have become more common. But even so you may be a little startled by the statement that the concrete industry is a billion dollar industry. In fact the total annual cost of concrete produced at the present time very likely considerably exceeds $1,000,000,000.

If we translate the dollar unit into terms of human effort or man power we might say that our industry is measured by the daily toil of an army of a million men. If these figures can be visualized they represent in a broad way out present dimensions.

By the time we celebrate another twentieth anniversary we shall undoubtedly more than double our present annual output which means that in the next twenty years we shall add to our wealth or the permanent investment in "plant" in concrete construction, a sum equal to our total national debt.

It is not my purpose to inflate our importance or stagger the imagination with huge figures, but rather to use this dimensional outline as a background for some remarks on the responsibility which I feel this Institute must assume with reference to the concrete industry.

With all due respect to other agencies that are at work in our field to solve our many problems, it seems to me that it is the special privilege and function of the Institute to assume and maintain leadership in the technical activities of the industry. While we have every reason to be proud of the work that has been done, of the service we have rendered in the past, the future value or usefulness of the Institute is intimately related to the seriousness with which we accept the responsibilities of our position.

It is not a casual matter to prepare and sponsor building regulations involving the expenditure of vast sums of money and affecting the economic status of the building industry.

It is not a mere incident to publish standard specifications for concrete structures and structural units. It is not a holiday jaunt to search out and combat successfully destructive agents affecting seriously the permanence of our concrete structures.

In fact, these are the most serious kinds of effort and our attitude toward and the manner in which we direct them will not only measure our service but will determine whether we shall maintain the leadership to which we aspire.

*President, American Concrete Institute, 1924-1926.

Concrete construction conserves our natural resources. It places at our disposal materials that are unlimited in quantity and universally distributed. We have no need to worry over the exhaustion of our sand and coarse aggregates deposits. It is quite true that we need fuel for the production of cement, form lumber to mold our concrete in and steel for reinforcement. But thanks to concrete construction in quantities of these materials are greatly reduced.

But, even if our raw materials are available in unlimited quantities these materials are transformed into structures only by the use of man power or labor, and labor is not available in unlimited quantities so we are forced nevertheless to consider economy and efficiency in the use of our materials.

One of our great national problems is the high cost of living. The high cost of construction is a large factor in this condition. We can help to reduce the cost of our goods if we reduce the cost of our manufacturing plants, of our warehouses where our goods are stored, our mercantile and office buildings where our business is transacted. Our taxes can be reduced if the cost of our public improvement is reduced. In so far as concrete is assuming a constantly growing importance in the building industry we are faced with the responsibility of lightening the burden of costly construction.

The revolt against the high cost of construction is reflected in attempts to standardize building methods, overhaul building codes, increase unit stresses, re-examine factors of safety, etc., while the inflation of unit stresses and infringement of the factor of safety may be decried as unnecessary and unwarranted in view of our vast wealth and resources. Still the general cry for economy demands an answer, and so far as concrete is concerned, the Institute should have a hand in framing the answer.

An increase of 2,000 lb. per sq. in. in the unit stress of the reinforcing steel would yield a saving of several millions of dollars per year. If we would produce a concrete having a compressive strength of 4,000 lb. per sq. in. instead of the 2,000-lb. concrete now in common use, additional millions of dollars per year could be saved. But these economics involve extensive investigations of engineering problems and a large program of tests of structural concrete members in the field and in the laboratory in order that proper factors of safety may be determined.

A general upward revision of unit stresses would also involve radical changes in existing building codes, which means the establishment of public confidence in the new state of affairs, a somewhat slow and difficult undertaking.

Yet to the extent that these questions can be answered the Institute should take an advanced position in finding a solution, by stimulating discussion, suggesting and planning research and co-ordinating the required effort.

Efficiency in the use of materials can only be attained by uniformity of product. In fact, the production of concrete of uniform strength and

dependable resistance to deterioration by the elements is the biggest single problem we have to deal with. The most serious criticism leveled at concrete as a structural material is its variability in strength. The utmost refinement in methods of design is of little assistance as a measure of economy if applied to materials whose physical properties cannot be controlled within reasonable limits of tolerance. Indeed, unit stresses, factors of safety and, in short, faith in the integrity of the structure will finally rest on the question of uniformity. We cannot afford to continue to use methods that leave the quality of the product within wide limits to chance.

As we look about us and examine the methods of concrete production, in general, we find that the really great advances in the art is the knowledge of the few rather than the common property of the many. The function of the Institute is to carry the message of making better concrete to the far corners of the earth.

Another problem the industry has wrestled with from its infancy is the satisfactory use of concrete as an architectural medium. It is natural to adapt a new material to old and recognized conceptions. The growth of new forms and methods of expression is usually slow and halting. While the great flexibility of concrete to adapt itself to almost any form has been recognized by architects for some time, difficulties in satisfactory treatment of the surface has retarded its general use and delayed the development and general adoption of concrete as a material having esthetic value.

Here again—the Institute should take up the "white man's burden" by inviting the pioneers in the field to bring in the fruits of their labor and experience in order that the architect, the artist and the artisan may take counsel with them and with each other as to what is good and may be retained and used and also as to what is bad and should be rejected.

The Institute work at the present, and for some time past for that matter, has reached a high degree of development. It is thoroughly coordinated and well directed. All the important subdivisions of industry are covered by committee activities.

There is, of course, more or less shifting of committee alignment and now and then new committees are added to take up the consideration of special subjects. But by and large, we seem to have reached a condition of stability or maturity in organization machinery. Does this mean that we have reached our full power in service and influence? Not by any means. The full benefit of the Institute work is derived only by those who take an active part in the work, those next benefited are the members who make use of the *Proceedings*, and still further out on the border of our influence are those who only occasionally draw from our store house of information. Now it is important to note that not much more than 10 per cent of the membership are actively engaged—these are the men who are playing the game—and they derive the greatest good—the balance are spectators but they benefit at least to the extent of observing personally how the game is going—the rest, the great general public, takes no

part whatever—the great multitude is uninformed and unconcerned as to what is going on.

If for the general public we substitute the vast number who are engaged directly and indirectly in the concrete industry we can realize the importance of vitalizing and extending the Institute work. The problem is to find a practicable method of converting a larger share of the potential energy of the 90 per cent that is now inactive. A solution to the problem is under consideration. The holding of between-convention regional meetings at points away from the usual centers of the annual convention is conceived to be a step in this direction. We must find some means of effectively distributing the material that is brought in annually to our storehouse. We are on the verge of some important and fundamental changes in our method of providing concrete. We have passed through a cycle of the early simple process of combining a unit quantity of cement with fixed proportions of fine and coarse aggregate to a more complex method of designing the mix on the basis of a screen analysis of the aggregate combined with a suitable water cement ratio, and are now about to emerge to an even simpler process than that with which we began, by merely specifying the quantity of water per unit volume of cement, allowing the proportions of the aggregate within reasonable limits to be determined by workability. I say we are about to emerge for we have not yet arrived at a point where this broad and simple conception can be pronounced as thoroughly practicable and satisfactory. Should the extensive investigations which are under way establish the soundness of the new practice we shall revolutionize present methods and enter a new era in the art of making concrete. Another and equally important "event is casting its shadow before."

For years we have struggled to control the variability in the strength of the concrete—we have added investigation to investigation and the end is not yet—but almost—in sight. Papers at this convention will indicate how nearly we are approaching the goal of uniformity in quality. In fact, it is predicted by competent authority—that it is reasonable to expect that concrete can be made the most reliable and uniform building material we have.

The temptation is strong to expand upon the theme of the advances that have been made and those that are about to be made, but we need only glance over the past in order to gather confidence in the future.

The Institute in common with the Industry in general has passed beyond the Pioneer Stage. In the building of the road to better concrete—the reconnaissance work through much of the unknown country has been completed. An earlier generation blazed the trail through the trackless wilderness. The early tracks were often tortuous and indistinct, considerable relocation and straightening of the road has been required—we have had a good deal of pulling of stumps and blasting of rocks, leveling of hills, and filling of hollows. The subgrade has been prepared and now we are engaged in the building of the permanent weaving surface. Some

of the earlier work may have to be replaced, but the bulk of our new construction seems to be completed. Our road is gradually taking on a more finished appearance and we look forward hopefully to the time when the completed project may efficiently and satisfactorily serve the needs of our civilization.

Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders.

By H. M. Westergaard.*

The reinforced-concrete structure consisting of square or rectangular slabs each of which is supported on four sides on beams or walls possesses obvious architectural advantages, and deserves for this reason alone a careful examination of its features of strength and economy. The problem involves a consideration of the slabs and of the supporting girders.

Six types of panels should be considered; they are represented, for example, in Figs. 1 and 2: (1) the single panel; (2) end panels of a row of panels; (3) intermediate panels in a row; (4) corner panels; (5) wall panels; and (6) interior panels. The analytical and experimental material which is available does not cover completely this diversity of cases. Some of the missing cases, however, are so decidedly intermediate between cases about which information can be found that it seems to be possible to make reasonable estimates for them. At the same time, further investigations are desired, dealing, especially, with the missing cases, in order that the questions may be settled in a more than provisional manner.

The present study is based on information of the following four kinds:

(1) Results obtained by the theory of elasticity.

(2) Results of tests of slabs of the kind dealt with here.

(3) General information concerning the phenomenon of redistribution of stress resulting from redistribution of relative stiffness as the stresses increase.

(4) Knowledge of the behavior of flat slabs.

Figs. 1 and 2 show coefficients of bending moments per unit of width, obtained by use of the theory of elasticity. The slabs are assumed here to be of homogeneous, isotropic, elastic material with Poisson's ratio equal to zero. Each panel is simply supported on four sides on rigid beams, but the slab is continuous over the interior beams. Each shaded panel carries a total uniform load W. Each unshaded panel is entirely unloaded. The elements of section which are indicated on the drawings, and to which the coefficients apply, unless they are diagonal elements at corners, belong to total sections each of which extends across a panel along a central line of symmetry (with positive moments) or along an edge (with negative moments). In terms of the coefficients q stated in Figs. 1 and 2, the

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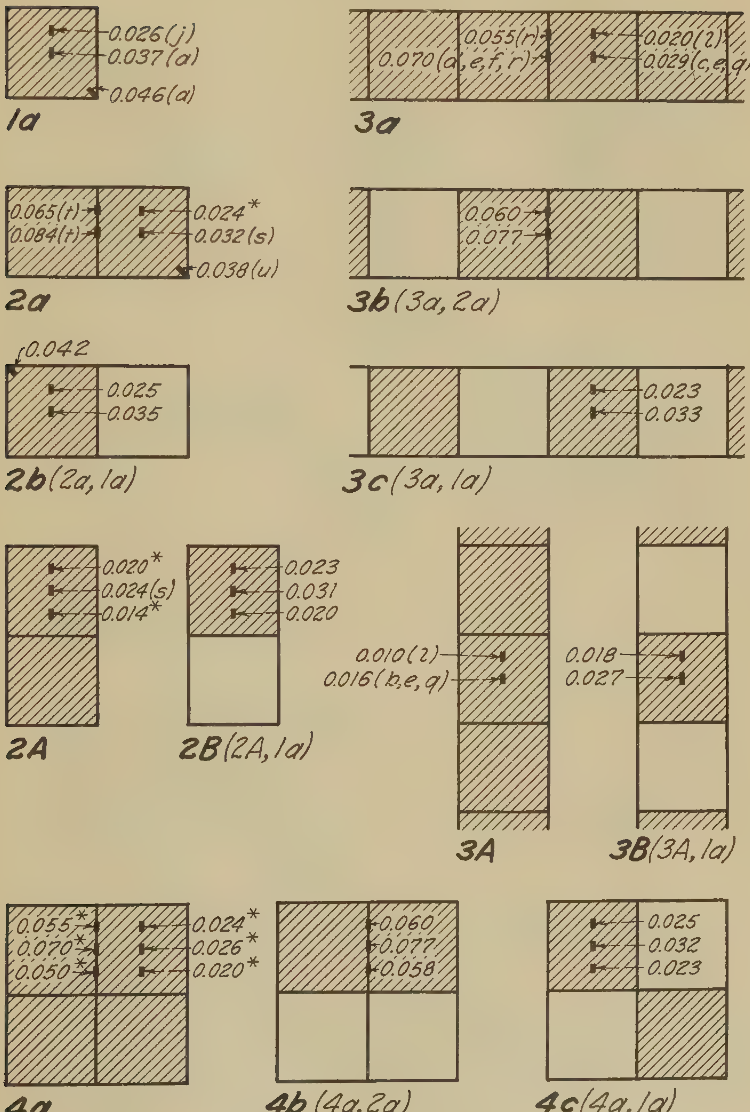

FIG. 1.—COEFFICIENTS OF BENDING MOMENTS PER UNIT OF WIDTH AT VARIOUS POINTS IN SQUARE PANELS OF HOMOGENEOUS, ISOTROPIC, ELASTIC MATERIAL WITH POISSON'S RATIO EQUAL TO ZERO.

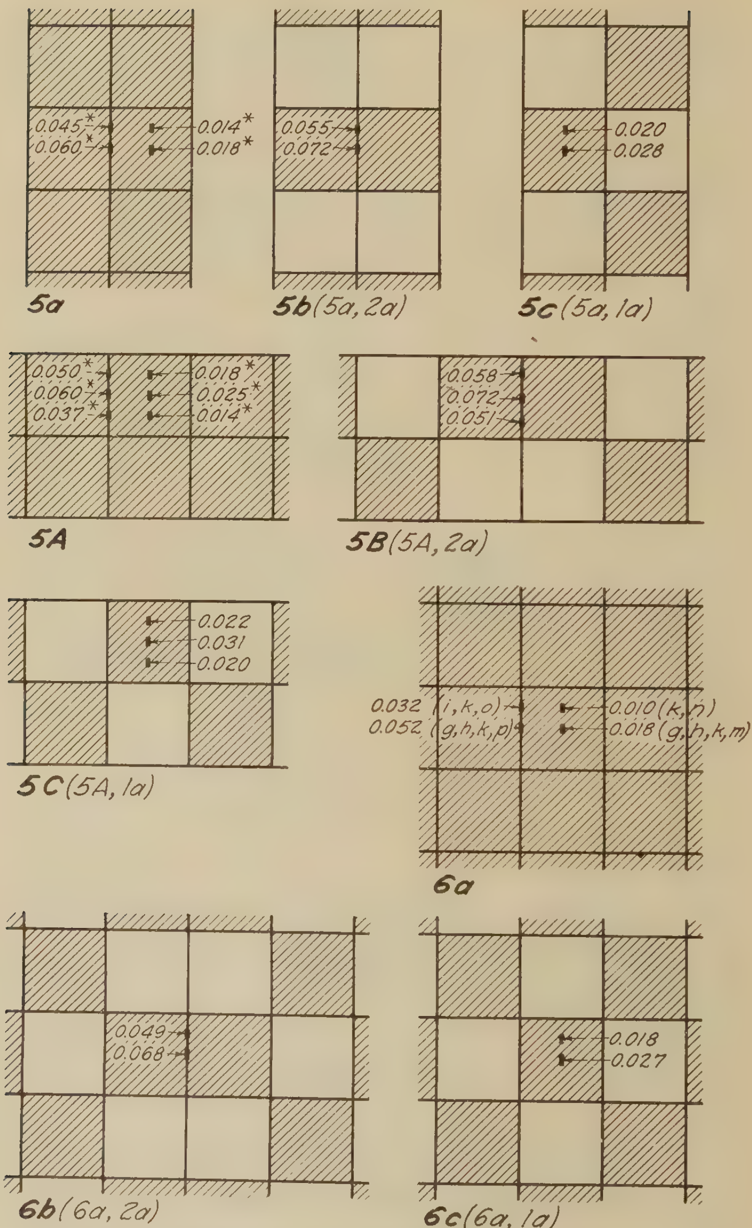

FIG. 2.—COEFFICIENTS OF BENDING MOMENTS AS IN FIG. 1.

bending moments per unit of width of section may be computed by the formula

$$M = \pm qW. \tag{1}$$

With W stated in pounds, this formula gives the bending moment in pounds. or in inch-pounds per inch width of section.

If Poisson's ratio is changed from zero to some positive value μ, the bending moments M and M_y at any given point, in the directions of x and y (that is, in elements of section parallel to y and x, respectively), will obtain, in all the cases dealt with here, the following values:

$$M'_x = M_x + \mu M_y,\ M'_y = M_y + \mu M_x \tag{2}$$

respectively. Along any edge in the direction of y, one finds $M_y = 0$, that is, according to equations (2), $M'_x = M_x$. The coefficients q for negative bending moments at the edges, consequently, apply independently of the value of Poisson's ratio. When Poisson's ratio is changed from zero to a positive value μ, the positive moments at the centers, on the other hand, are increased, while the diagonal moments at the corners are multiplied by $1 - \mu$.

With values of M'_x and M'_y known for a given value of μ, one finds by solving equations (2):

$$\begin{aligned} M_x &= \frac{M'_x - \mu M'_y}{1 - \mu^2} \\ M_y &= \frac{M'_y - \mu M'_x}{1 - \mu^2} \end{aligned} \tag{3}$$

These formulas were used in a number of the cases in Figs. 1 and 2 in computing coefficients q for M_x and M_y on the basis of values of M_x' and M'_y given by different investigators for positive values of μ.

The letters given in parentheses in Figs. 1 and 2, next to the coefficients, refer to the following sources, from which the values of the coefficients have been taken (in some cases through application of equations (3)):

(a) These coefficients have been computed by a number of investigators. A discussion of the values may be found on p. 433 in the paper by *W. A. Slater* and the writer, Moments and Stresses in Slabs, American Concrete Institute, *Proceedings*, v. 17, 1921, pp. 415-538, (or, National Research Council, Reprint and Circular Series, No. 32).

(b) Ibid., p. 432, Fig. 4.

(c) Ibid., p. 433, Fig. 5.

(d) Ibid., p. 434, Fig. 6.

(e) *A. Nadai,* Die Formanderungen und die Spannungen von Rechteckigen Elastischen Platten, Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, 170-171, 1915 (87 pp.), p. 62.

(f) *A. Nadai*, Die Elastischen Platten (Berlin), 1925 (326 pp.), p. 134.

(g) Ibid., p. 184.

(h) *H. Hencky*, Der Spannungszustand in Rechteckigen Platten (Munich and Berlin), 1913 (94 pp.), p. 53.

(i) Ibid., Plate V.

(j) *H. Leitz*, Die Berechnung der Frei Aufliegenden, Rechteckigen Platten (Berlin), 1914, p. 27.

(k) *H. Leitz*, Berechnung der Eingespannten Rechteckigen Platte, Zeitschrift für Mathematik und Physik, v. 64, 1916, pp. 262-272, p. 270.

(l) *N. J. Nielsen*, Bestemmelse af Spaendinger i Plader ved Anvendelse af Differensligninger (Copenhagen), 1920, (232 pp.), p. 153 (approximate values).

(m) *B. G. Galerkin*, paper in Russian on Rods and Slabs, Vestnik Engeneroff, 1915, No. 19, reprint, p. 29.

(n) Ibid., pp. 29, 31.

(o) Ibid., p. 33.

(p) Ibid., pp. 29, 35.

(q) *B. G. Galerkin*, paper in Russian on Flexure of Rectangular Slabs and Thin Walls, Bulletin of the Polytechnic Institute of Petrograd (Leningrad), 1916, pp. 172-173.

(r) Ibid., p. 176.

(s) Ibid., pp. 194-195.

(t) Ibid., p. 198.

(u) Ibid., pp. 206-207.

(*) Estimated value.

In case 2b in Fig. 1 the left panel is assumed to be loaded, and the right panel unloaded. If, instead, the left panel were loaded by W and the right panel by $-W$, the common edge would be a line of anti-symmetry for the deflected middle surface, and the bending moments at this edge in the direction perpendicular to the edge would be zero. The left panel, then, would deflect like the single panel in case 1a. By superimposing the load W, $-W$ for the two panels on the load W, W (case 2a), one obtains the resultant load $2W$ for the left panel, and zero for the right panel. The bending moments in the loaded panel in case 2b may be obtained, therefore, by taking the average of the corresponding bending moments in cases 2a and 1a. This averaging is indicated on the drawing by the notation (2a, 1a). The coefficients stated for case 2b evidently represent maximum values of the positive moments in the particular elements of section.

It has been possible to apply the same scheme of averaging throughout Figs. 1 and 2 for the purpose of determining the coefficients of the greatest possible moments. For example, case 4b is dealt with by taking the averages of the values in cases 4a and 2a. In three of the cases, 3b, 5B, and 6b, the averages which are indicated by the notation, and which were used in computing the coefficients, correspond not exactly, yet with a quite satisfactory approximation, to the case of loading which is indi-

cated by the shading, and which leads to the true maximum values. In case 3b, for example, by taking the averages of values from cases 3a and 2a, one obtains coefficients which apply strictly to the arrangement . . 0, *W*, *W*, 0, 0, *W*, *W*, 0, 0, *W*, *W*, 0 . . of loads on the panels in the row instead of the arrangement . . 0, *W*, 0, *W*, *W*, 0, *W*, 0. . . which is indicated by the shading. The errors resulting from differences of this kind in the three cases can not possibly be very large.

The coefficients of moments which are stated in Figs. 1 and 2, and which apply, presumably, to homogeneous elastic material, may be used as a guide in establishing formulas for the design of slabs of reinforced concrete, with the same pattern of panels. It is understood that these design formulas are to be used in computing the amounts of steel on the basis of the same nominal working stresses that are used in case of beams. If the same formulas are used in computing nominal stresses in the concrete, it is possible that working stresses should be used for the concrete which are different from those used in the case of beams. Since the working stresses are merely nominal stresses, used in computations to attain safe and economical design, it is to be expected that the coefficients shown in Figs. 1 and 2 will call for some revision before they are applied to slabs of reinforced concrete. This revision must be based on what is known from tests and experience concerning the behavior of the slabs and the materials.

Tests of slabs of the kind dealt with here were mentioned as one of the sources of information. Some tests of this kind were discussed by W. A. Slater and the writer in a paper published in the *Proceedings*, 1921.* They are tests made under the direction of *Bach* and *Graf*,† and a test made at Waynesburg, Ohio, for J. J. Whitacre, under the direction of W. A. Slater. These tests indicate, generally speaking, the feasibility of using smaller coefficients than those obtained for homogeneous elastic material by use of the theory of elasticity.

The tests demonstrate quite clearly the phenomenon of redistribution of stress. This phenomenon may be explained in terms of what happens in the case of a rectangular interior panel. The load is assumed to be distributed uniformly over the whole floor and to be increased gradually from zero. With the smallest loads the panel will act most nearly like a homogeneous elastic slab. The greatest stresses will occur at the centers of the longer edges. The diagram of coefficients of bending moments across the edge shows small values near the corners and large values at the middle. When the load is increased, the increase of bending moment corresponding to a given increase of deformation will become smaller at the middle of the edge where the stresses are large than near the corners where the stresses are small. That is, when the load increases, the stiffness of the material becomes relatively smaller at the middle of the edge than near the corners. The result is that the parts near the corners become relatively more active. One may say that stresses are thrown from the

* W. A. Slater and the writer, Moments and stresses in slabs, American Concrete Institute, Proceedings, v. 17, 1921, pp. 415-538, especially, pp. 487-500 (or, National Research Council, Reprint and Circular Series, No. 32).
† Deutscher Ausschuss für Eisenbeton, v. 30, 1915.

middle of the edge toward the corners, and the middle borrows strength from the sides. Accordingly, the diagram of coefficients of bending moments is flattened out, the ordinates becoming smaller at the middle and larger near the ends. At the same time, the fact that the deformations are smaller at the middle of the panel than at the longer edges, causes a second redistribution of the coefficients of bending moments, or borrowing of strength, with decreases of the negative moments at the edges and increases of the positive moments at the middle of the panel. When the stresses near the middle increase, a third redistribution occurs; the diagram of positive moments across the longer line of symmetry is flattened out. Finally, there is a redistribution of action from the shorter span to the longer span. These redistributions result, generally speaking, in a lowering of the critical coefficients by which the slab would be designed.

One finds an argument in favor of counting on this lowering of the critical coefficients in what is known generally concerning the phenomenon of redistribution of stress or borrowing of strength in ductile materials. The theory of elasticity may indicate, for example, a concentration of stress at the edge of a rivet hole. But tests with loads beyond the proportional limit have shown in many cases of this sort that local yieldings will redistribute the stresses more nearly uniformly, with the result that the critical stresses become somewhat lower than is indicated by direct application of the theory of elasticity.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete in the report of 1924, in Section 142, specifies a coefficient of 0.09 for the sum of positive and negative moments in a flat slab. The corresponding coefficient derived by applying principles of statics to a uniformly loaded slab is 0.125. The many tests of flat slabs have indicated the permissibility of the reduction from 0.125 to 0.09 in this coefficient, by which the nominal stresses are computed for the purpose of design. The flexure of the slab supported on girders in two directions is similar in many ways to the flexure of the flat slab. There is reason to assume, therefore, that a reduction of the coefficients of moments in the ratio of 0.09 to 0.125, that is, a reduction amounting to 28 per cent, is permissible in the case of square panels supported on four sides.

Fig. 3 shows a set of coefficients proposed for the use in design of square panels supported on four sides. The method of using the coefficients is explained in the note under the figure. These coefficients have been obtained by using the coefficients in Figs. 1 and 2 as a guide, by taking into account the redistributions of stress, by taking into account also the improbability of some of the cases of loading in Figs. 1 and 2, and finally by permitting, generally speaking, a reduction of the coefficients to the extent of 28 per cent. The coefficients have been obtained under the assumption that the deflections of the girders are small compared with the deflections of the central portions of the loaded panels. If the girders are slightly less rigid than they should be according to this assumption, the effect may be assumed to be mainly an increase of the stresses in the side strips, and the coefficients, probably, need not be changed.

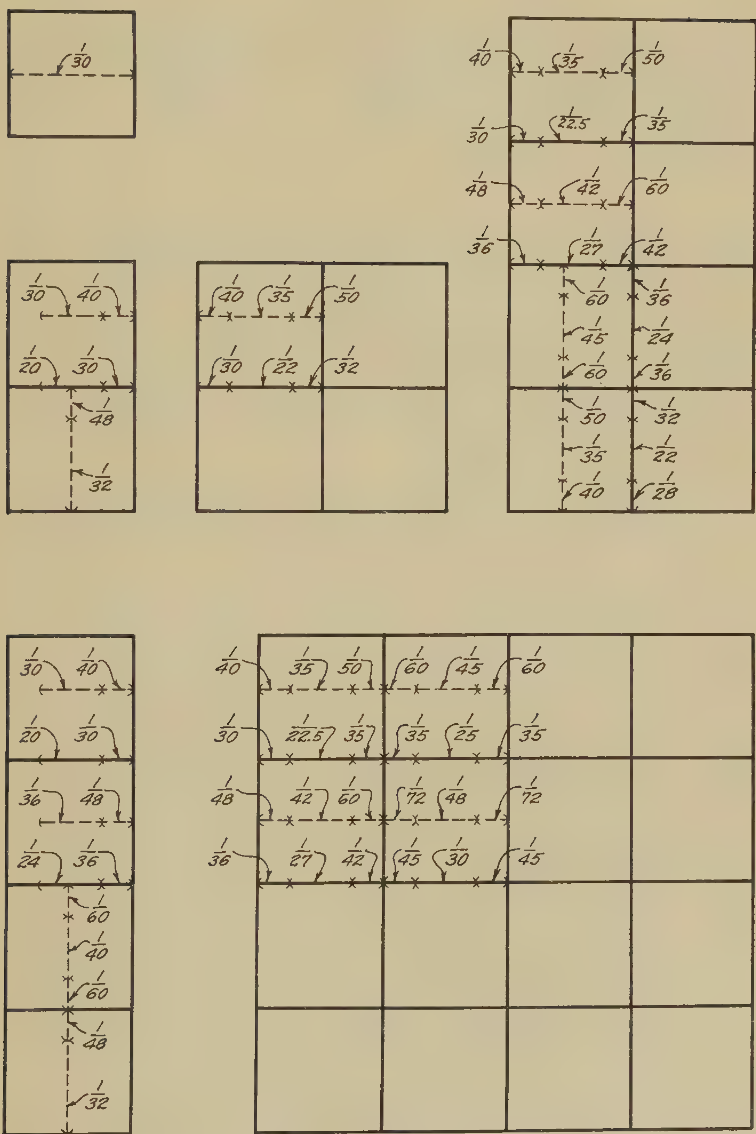

FIG. 3.—PROPOSED COEFFICIENTS OF BENDING MOMENTS FOR THE DESIGN OF SQUARE PANELS OF REINFORCED CONCRETE.

$M = \pm qWb$ = bending moment in the strip of width b; q = coefficient given by the numbers; W = total uniform load on one panel. The width of the side-strips is one-fourth of the span.

Figs. 4 and 5 show sets of similar coefficients proposed for rectangular panels. The use of the coefficients is explained in the note under Fig. 4. The bending moments in the shorter span are expressed in terms of the ratio m of the shorter span divided by the longer span by formulas of the type

$$M = \pm \frac{c_1}{1 + c_2 m^3} w l^2 b, \qquad (4)$$

where c_1 and c_2 are numbers. With $m = 1$, corresponding to square slabs, the coefficients assume the values given in Fig. 3. With $m = 0$, corresponding to one-way slabs, the coefficients assume the values which are used ordinarily in this case. The form of equation (4) agrees satisfactorily with the results found for rectangular panels by the theory of elasticity.*

Fig. 6 shows a set of coefficients which have been computed for the girders in the case of square panels. The shaded panels are loaded, the unshaded panels entirely unloaded.

The computation may be explained by referring, first, to case 1a. The structure is assumed to be simply supported at the four corners. In a cross-section through the slab and the girders along the line of symmetry, the total bending moment in the slab and the girders combined is $\frac{1}{8} W l$. Whatever portion of this bending moment is not accounted for in the slab, must be found in the two girders. According to Fig. 3 the slab is to be designed for a nominal bending moment equal to $\frac{1}{30} W$ per unit of width. This nominal bending moment represents, presumably, a true bending moment which is defined by the condition that the nominal moment is obtained from it by a reduction of 28 per cent. The true bending moment thus accounted for is $\frac{1}{0.72} \times \frac{1}{30} W$ per unit of width at the center of the slab. The bending moments in the slab are so distributed over the cross-section that the average value is about 0.70 times the maximum value. Accordingly, the total bending moment in the slab becomes $\frac{0.70}{0.72} \times \frac{1}{30} W l = 0.0324 \ W l$. The bending moment left for the two girders is

$$(0.1250 - 0.0324) \ W l = 0.0926 \ W l,$$

That is, the bending moment in each girder is

$$M = \tfrac{1}{2} \times 0.0926 \ Wl = 0.0463 \ Wl,$$

as is indicated in Fig. 6.

* See for example, W. A. Slater and the writer, l.c., Figs. 3-10, pp. 431-438.

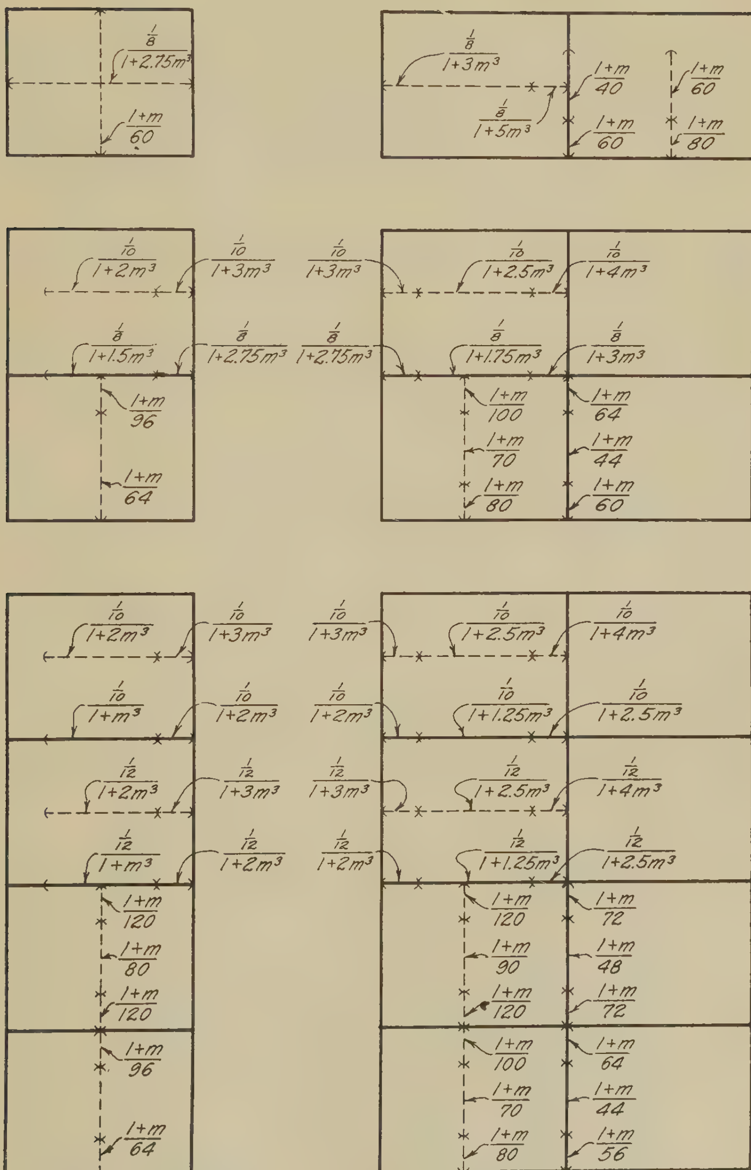

FIG. 4.—PROPOSED COEFFICIENTS OF BENDING MOMENTS FOR THE DESIGN OF RECTANGULAR PANELS.

$M = \pm q\, wl^2b$ = bending moment in a strip of width b; q = coefficient given by the formulas in terms of the ratio m of the shorter span, l, divided by the longer span; w = uniformly distributed load per unit of area. The width of the side-strips is one-fourth of the shorter span.

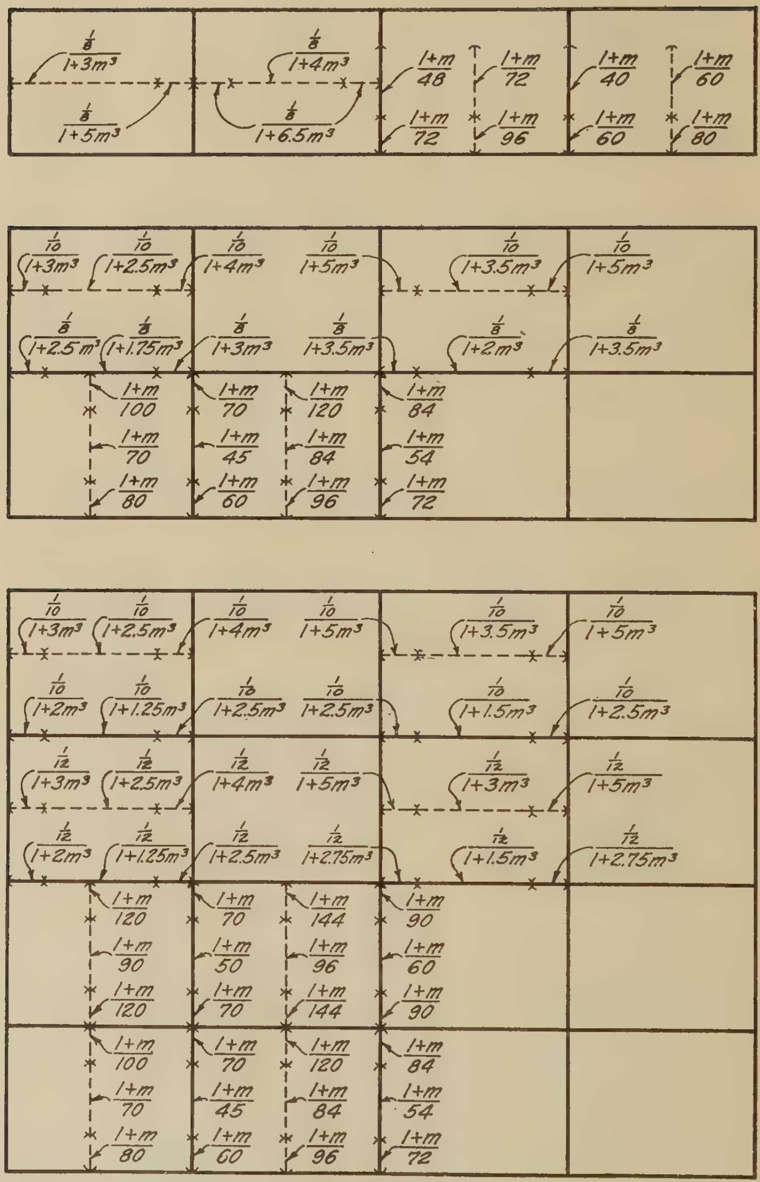

FIG. 5.—COEFFICIENTS OF BENDING MOMENTS AS IN FIG. 4.

The fact that this bending moment is rather large may be explained through the tendency of the corners to curl up. This curling up is assumed to be prevented by the design of the structure. The forces holding the corners down may be interpreted as negative reactions. Their presence causes the downward load on each girder between the supports to be larger than one-fourth of the load on the panel.

For a second illustration take case 4A. Each girder is a continuous beam with two spans of length l and three simple supports. Let axes of co-ordinates x and y be placed along the two inside girders. Two cross-sections are passed along the lines $x = \frac{l}{2}$ and $x = 0$ through slab and girders. By use of simple principles of statics it is seen that the sum of the positive bending moments in the section $x = \frac{l}{2}$ in the two panels of slab and the three girders, plus one-half the sum of the negative moments in slab and girders in the section $x = 0$ must be equal to $\frac{1}{8}$ $(2W)\ l$. In Fig. 3 the coefficient of the greatest positive nominal bending moment is given as $\frac{1}{35}$. As is indicated in Fig. 1 for case 4c, the greatest positive moment is obtained by averaging the values in cases 4a and 1a. Thus the nominal value $\frac{1}{35}$ should be the average of the coefficient of the nominal bending moment for case 4a and the value $\frac{1}{30}$ for case 1a. That is, one finds for case 4a in Fig. 1 the nominal coefficient of positive moments $\frac{2}{35} - \frac{1}{30} = \frac{1}{42}$. In the same way, one finds for the negative moments in case 4a: $\frac{2}{22} - \frac{1}{20} =$ approximately $\frac{1}{24}$. By using the factor 0.72 in the same manner that was described for case 1a in Fig. 6, and by using the ratio 0.65 of the average to the maximum value instead of the ratio 0.70 used in case 1a, one obtains the following value of the sum of the positive moments in the slab in the section $x = \frac{l}{2}$ plus one-half the sum of the negative moments in the section $x = 0$:

$$\frac{0.65}{0.72}\left(\frac{1}{42} + \frac{1}{2} \cdot \frac{1}{24}\right) W\ 2l = 0.0806\ W\ l.$$

The corresponding sum left for the three girders is, then,

$$(0.2500 - 0.0806)\ W\ l = 0.1694\ W\ l.$$

If the load on each girder were distributed uniformly, the ratio of the positive moment at the center of the span to the sum of the positive moment at the center plus one-half the negative moment at the middle support would be 0.5. With the distribution of loads found here this ratio will be, instead, approximately, 0.54. Accordingly, the sum of the

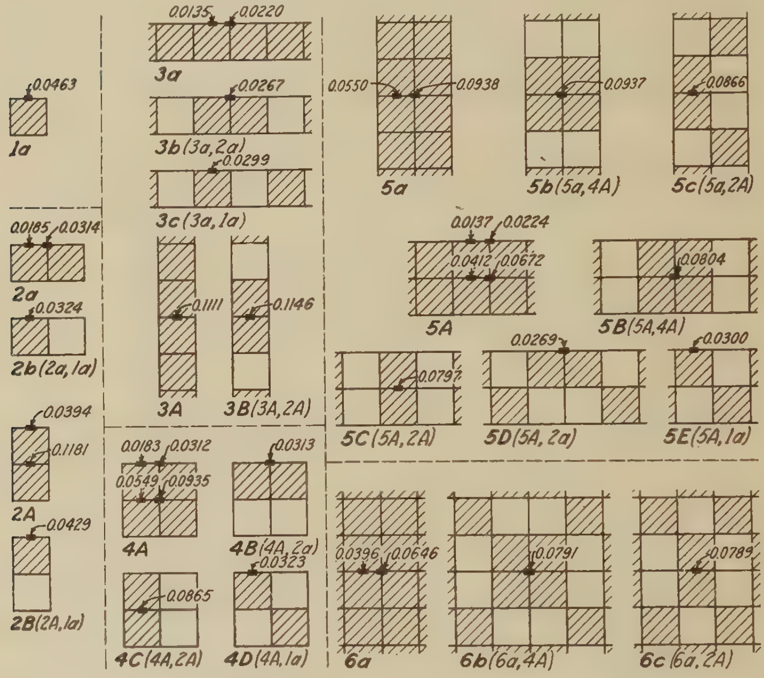

FIG. 6.—COEFFICIENTS OF BENDING MOMENTS COMPUTED FOR THE GIRDERS IN THE CASE OF SQUARE PANELS.

The girders are simply supported, but continuous over the intermediate supports. Moment $M = \pm qWl$; $q =$ coefficient; $W =$ uniform load on one panel; $l =$ span. The sections are at the centers and at the ends of the spans.

positive moments in the three girders in the section $x = \dfrac{l}{2}$ becomes

$$0.54 \,.\, 0.1694\, W\, l = 0.0915\, W\, l,$$

and the corresponding sum of the negative moments in the three girders in the section $x = 0$ becomes

$$2\,(1 - 0.54)\,.\, 0.1694\, W\, l = 0.1558\, W\, l.$$

Each of the outside girders receives approximately one-fifth of each of these two total moments, whereas the inside girder receives three-fifths. One finds then the following moments:

Outside girder, positive moment at the center of the span:

$$0.2 \;.\; 0.0915\, W\, l = 0.0183\, W\, l.$$

Outside girder, negative moment at the intermediate support:

$$0.2 \;.\; 0.1558\, W\, l = 0.0312\, W\, l.$$

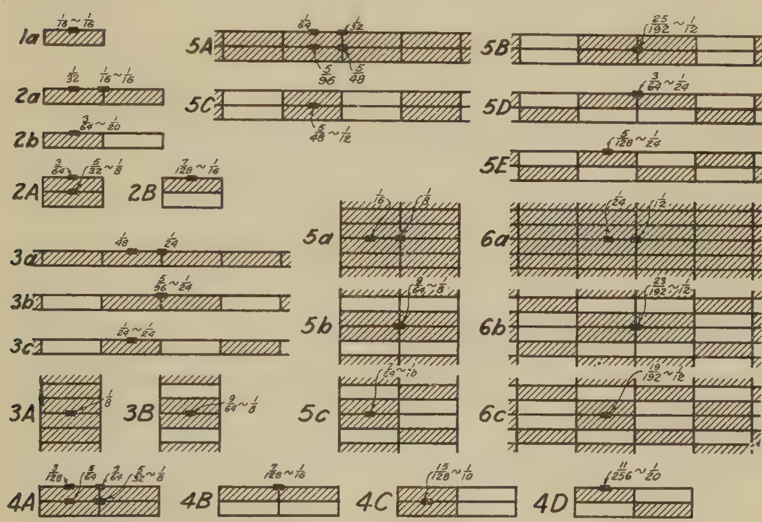

FIG. 7.—COEFFICIENTS OF BENDING MOMENTS IN THE GIRDERS IN THE CASE OF A ONE-WAY SYSTEM.

The girders are simply supported, but continuous over the intermediate supports. When two coefficients q are stated, the first is the value resulting from computation, the second is the value which would be used in design. The bending moment is expressed as $M = \pm\, qWl'$; $W =$ uniform load on one panel; $l' =$ span of the girder. The sections are at the centers and at the ends of the spans.

Inside girder, positive moment at the center of the span:

$$0.6 \;.\; 0.0915\, W\, l = 0.0549\, W\, l.$$

Inside girder, negative moment at the intermediate support:

$$0.6 \;.\; 0.1558\, W\, l = 0.0935\, W\, l.$$

The four coefficients obtained here are stated in Fig. 6 for case 4A.

All of the coefficients in Fig. 6 for the cases of full load on all panels have been obtained by the method which has now been shown for the two cases 1a and 4A. For the ratio of the average bending moment in a cross-section in the slab to the maximum bending moment in the same

section the ratio 0.70 was used in the cases 1a, 2a, and 3a; 0.65 was used in the cases 2A, 4A, and 5A; and 0.60 was used in the cases 3A, 5a, and 6a. For the ratio of the sum of the positive moments at the centers of the span to the sum of these positive moments plus one-half the sum of the negative moments for both ends of the same spans the following values were used:

Case:	1a	2a	2A	3a	3A	4A	5a	5A	6a
Ratio:	1	0.54	1	0.38	1	0.54	0.54	0.38	0.38

Fig. 6 shows a number of cases in which some of the panels are loaded and others unloaded. These arrangements of loads lead to the greatest possible values of the moments at the points for which coefficients are stated. In these cases the same scheme of averaging was used for the girders as was used for the slab in Figs. 1 and 2. The scheme is based on the principles which were explained in connection with Figs. 1 and 2. The notation by which the two components of each average are indicated is the same as is used in Figs. 1 and 2. For example, the coefficient stated for case 3b is the average of the corresponding values for cases 3a and 1a. As in Figs. 1 and 2, the averages correspond only approximately to the distribution of the load indicated by the shading in some of the cases; namely, in Fig. 6, in cases 3b, 3B, 5b, 5c, 5B, 5D, 6b, and 6c. The approximation, however, is satisfactory for the present purpose, although it is in some of the cases less close than was obtained in Figs. 1 and 2.

Fig. 7 shows a set of coefficients corresponding to those given in Fig. 6, but for a one-way system of slabs and girders. In the cases of partial loading the coefficients were obtained as averages which correspond exactly to the averages in Fig. 6.

Fig. 7 shows also those coefficients which would be used in design. In some of the cases the coefficients used in design are considerably smaller than those computed. The reductions involved here may be justified on the ground of the improbability of some of the cases of loading, and on the ground that the inside girders, for which the reductions are the greatest. are less subject to torsion than are the outside girders.

It seems to be reasonable that reductions of the kind permitted in the cases in Fig. 7 should be permitted also in the cases in Fig. 6. It may be noted in this connection that the errors incurred by the averaging of coefficients in the cases in which this method is only approximate are of the same kind in Fig. 6 as in Fig. 7. The reductions indicated by comparing the two sets of coefficients in Fig. 7 should be suitable, therefore, as a guide in deciding on corresponding reductions in the cases in Fig. 6.

Use of Figs. 6 and 7 in this manner led to the coefficients which are obtained in Figs. 8 and 9 by substituting $m = 1$, corresponding to square panels, or $m = 0$, corresponding to one-way systems.

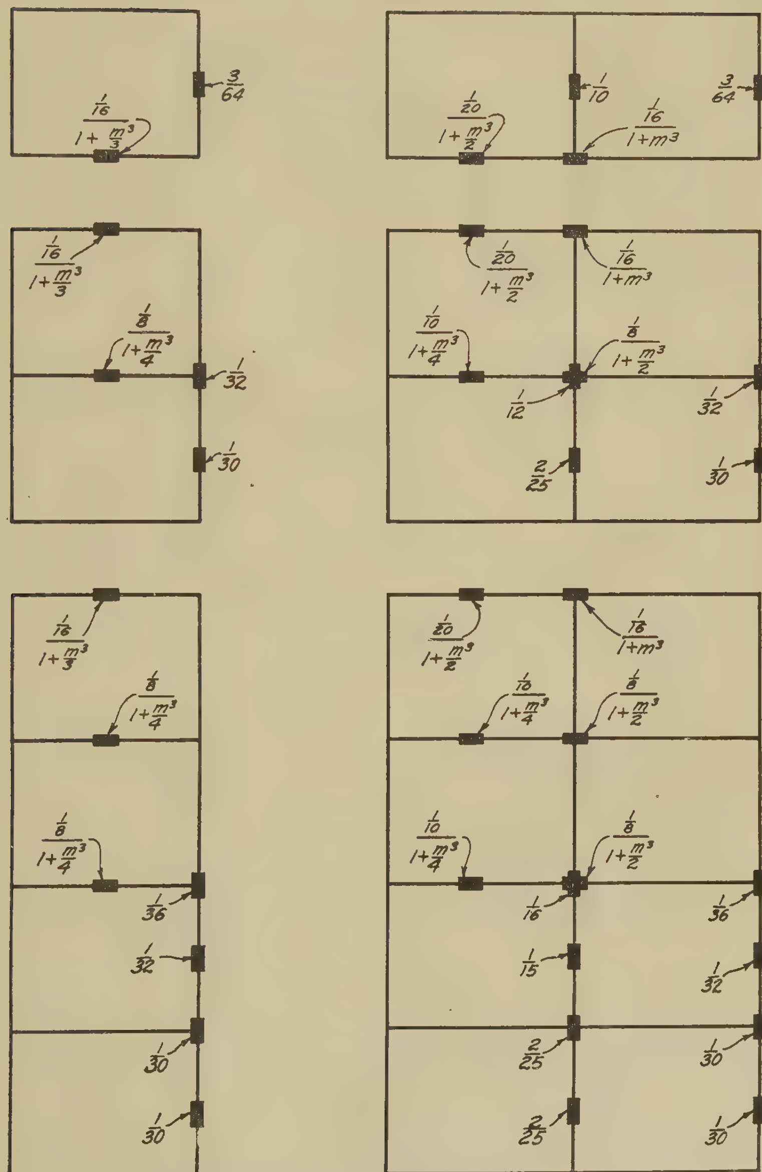

FIG. 8.—PROPOSED COEFFICIENTS OF BENDING MOMENTS FOR THE DESIGN OF THE GIRDERS.

The girders are simply supported at the corners of the panels only, and are continuous over the intermediate supports. Moment for the longer span: $M = \pm qWl'$; for the shorter span: $M = \pm qwl^3$; q = coefficient; W = total uniform load on one panel; w = load per unit of area; l' = longer span; l = shorter span; $m = l/l'$.

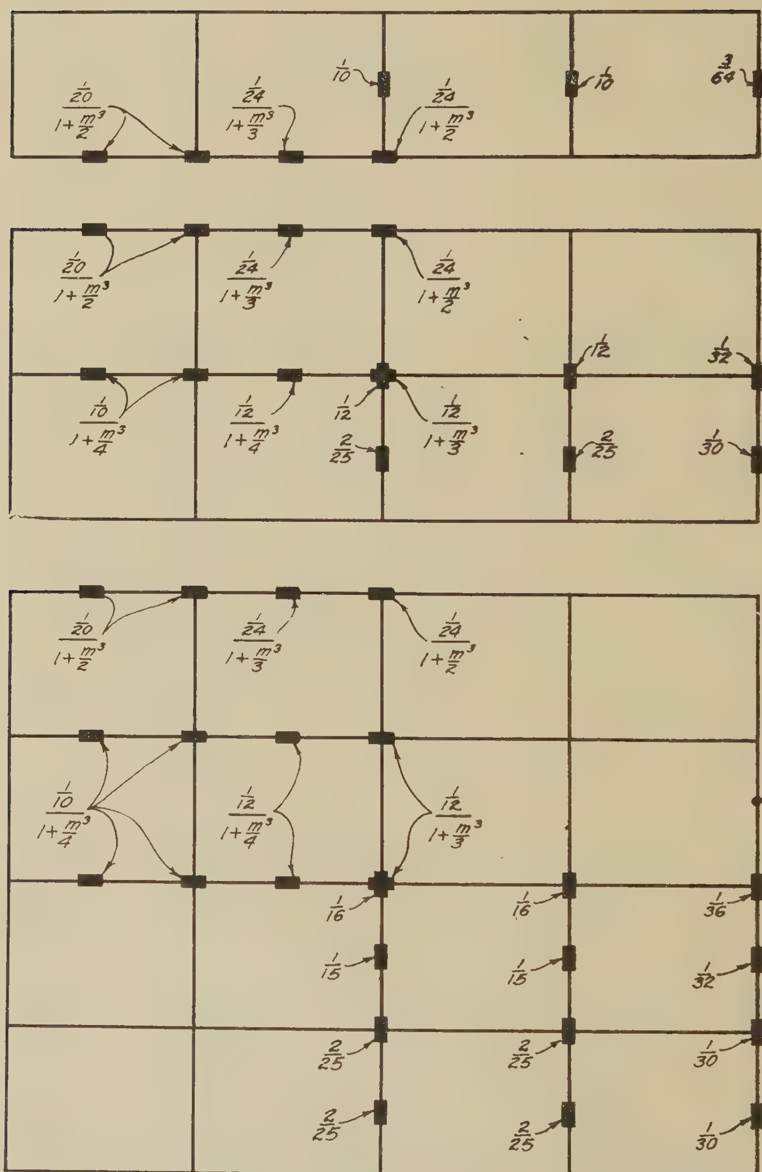

FIG. 9.—COEFFICIENTS OF BENDING MOMENTS AS IN FIG. 8.

Figs. 8 and 9 show the coefficients proposed for the girders in the case of rectangular panels. The formulas given in Figs. 8 and 9 for the coefficients of moments for the longer span are of the same general type as those given in Figs. 4 and 5 for the shorter spans of the slabs. The use of the coefficients is explained in the note under Fig. 8.

DISCUSSION.

Mr. Magee. C. L. C. Magee—I am making quite a study of just this situation, and I do not think enough attention so far has been given to what unquestionably are secondary supporting stresses in concrete slabs that are functions of deflection. As a slab deflects, we get circumferential stresses; the slab crowds toward the center. Those are not radial stresses, neither are they exact stresses, they are circumferential. If you analyze a flat slab or a square panel or a rectangular slab on the basis of circular plates and consider the pointups invection in a circular plate, you get an entirely different set of conditions. I would agree with Dr. Westergaard that about one twenty-fourth would be a proper measurement on the two-way slab, but if you take those other factors into consideration, about one thirty-second is pretty nearly mathematically correct.

Mr. Lindau. A. E. Lindau—What did you find to be the moment coefficient in the uniform load of a square panel, for example? Just approximately?

Prof. Westergaard. Professor Westergaard—Without any reduction, about one over twenty-four; that is, the bending moment at the middle can be computed as one over twenty-four times the total load on the panel times the width of the strip which is considered.

Mr. Lindau. Mr. Lindau—That would ordinarily be considered if you'd distribute the load in two directions as a sixteenth, so this gives it that additional factor of the difference between a sixteenth and a twenty-fourth.

Prof. Westergaard. Professor Westergaard—Then this provision may be modified by a reduction of the same type which is used in the case mentioned. The value I indicated is one-thirtieth in this particular case; the reduction is not quite 28 per cent.

Permanent Railroad Track.

By Frank N. Alfred.*

Civil engineering was, in the nineteenth century, largely confined to the location and building of railroads. The actual maintenance of the property, when once constructed, likewise the equipment purchased, was entrusted to non-technical men. That era in this country is rapidly coming to an end. We are entering into an era of refinement—it may be called the age of highly developed usefulness, in which the phenomena of nature will be solved and understood, and developed by persevering minds that will not be content to waste away while the great crowd drifts wherever the current carries.

Historical Aspects.—The first railroad (if such it may be called) was built in Massachusetts. It was an industrial tramway running from Bryant quarries, about three miles to tidewater at Nepenset, and used to transport blocks of granite by horse power. The construction consisted of granite blocks placed about 8 ft. apart, on which were placed wooden sleepers 1 ft. deep and 6 in. wide, on the top of which flat strips of iron 3 in. wide and ¼ in. thick were laid and spiked to the wood. It was operated by horse power. On the other hand, the first steam engine was run on the *highway.*

Various gauges from 3 ft. 4 in. to 6 ft. were experimented with. The gauge of railroads in England was fixed by Parliament in 1830 at not less than 4 ft. 8 in. between insides of rail, following the practice used in the building of the Stockton and Darlington Ry., which was begun May 13, 1822.

The T-rail was designed by Robert L. Stevens and first made in England and laid in this country on the Camden and Amboy road in 1831. He also designed the hook head spike. Wrought iron step chairs were used under the rail joints.

The steam railway age in America was as feverish from 1830 to 1900 as have been the automobile and good roads development in the first quarter of the twentieth century. State ownership was resorted to by Pennsylvania in the building of the Philadelphia & Columbia Ry. when in 1829 it took over the charter of Col. John Stevens. This practice was followed by other states. The states soon tired of the experiments and turned the roads over to private capital to operate.

The railroads were extended with such rapidity that by 1880 there were 93,671 miles in operation. The high tide was in the decade immediately following, when 65,000 miles of road were built,—seventy per cent as much as had been constructed in the previous half century.

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*President and General Manager, Pere Marquette Railway Co.

The *check* in expansion followed the creation of the Interstate Commerce Commission in 1887. That check was timely. It has since developed that many miles were built where there was not sufficient justification, and even today there exists much railroad that is unprofitable.

Henry V. Poor's Manual of 1868 gives the cost of building and equipping the early railroads as about $40,000 per mile in New England, the middle states roads at $53,000, the southern roads $30,000 and the western roads $41,000.

The early country highways cost from $300 to $500 per mile to build, turnpikes, $3,000 to $5,000. Today the hard surfaced highways cost from $30,000 to $50,00 per mile—as much as the railroads.

Present Status.—Railroads are using the same wood ties, with various kinds of ballast, track fastenings and T-rails as were used almost from the beginning. Is it strange that there is prejudice against the advance that has for its aim the doing away with these old friends of the railroad man? Did not the hard-road advocates have difficulty in securing its general adoption for heavy traffic, and have not their prophesies proven true?

I do not mean to speak disparagingly of the efforts that have been made in recent years to improve the track and roadbed of our railroads. There has been a very general and marked improvement in the riding qualities of track. Consideration is being given to better drainage of roadbed; heavier ballast is used; the life of ties and timber is being prolonged through treatment; crossing frogs and turnouts are being made with hard centers and of special steel; tie plates are being generally applied on tangents as well as curves, anti-creepers used to hold the rail to place, and heavier rail sections adopted. These improvements have not, in any perceptible measure, reduced the cost of track maintenance.

"Permanent Way" is a misnomer as applied to the standard track of the country today. The type of track in use requires minute daily inspection and constant work to keep it in line and surface. The weakest part of the track structure is the joint, and today engineers are not agreed upon the best type of joint; that is, as to whether the joint should be suspended or supported. Nor are they agreed as to whether the rail should be laid on a horizontal base or with a slight cant inward.

Much has been said about elasticity in track. The track of railroads is not rigid. Stand along the track and observe an approaching engine and note the wave motion created in the rail which is carried through the tie into the supporting ballast beneath. Resilience is essential as a compensating factor in transmitting the load over uneven surfaces such as exist in the present form of track. Whence the necessity for the proper equalizing through springs and other means of absorbing shock. I quote from Doctor Dudley, more than a quarter century ago:

> "I am inclined to think that, if the roadbed could be made absolutely unyielding, the springs of the vehicles providing the elasticity, the best results would be had. If the track could be as smooth and relatively as stiff as a planer bed, there would be a saving in the cost of maintenance of track and machinery, and in coal consumption. The stiffer the rails, the less the creeping due to the wave which runs ahead of the wheels, the less the wear of the ties due to this motion, the less the destruction of the track and running gear due to the pounding of the wheels and the easier the hauling of the trains."

The tracks of the D. L. & W. in the second Bergen Hill tunnel are carried on reinforced-concrete slabs. It was constructed in 1909 and seven years thereafter Lincoln Bush, who was chief engineer at the time of construction, said:

"My idea in getting out this design was if railroad track could be made perfectly rigid and unyielding that there would be no pounding or unusual stress in the rails. The roadbed in the new tunnel referred to has been in service under the heaviest kind of traffic since February, 1909, and has demonstrated fully that if track is made perfectly rigid it will stand up against the heaviest kind of traffic.

"I am convinced that with a perfectly rigid surface there will be no pounding and serious damage in railroad track unless some defect existed such as flat wheels. Such conditions, however, are corrected when they do arise with the railroad rolling stock. I have felt for some time that with the heavy rolling stock the best ballasted track had practically reached the limit of loading and if the heavy rail is used, the support of same on tie ballasted track is not like the abutments of a bridge, as ties yield, causing deflections in the rails, and it will be found that even with a heavier rail the stress may be greater in the rail than with a light and less stiff rail section which lends itself more readily to deflection."

O. E. Selby, in American Railway Engineering and Maintenance-of-Way Association Bulletin No. 80, said:

"Railroad track has grown in strength as heavier loads have made increased strength necessary, but such growth has been entirely along empirical lines and not one single detail of track superstructure bears marks of engineering design."

Many roads have adopted a permanent track in tunnels, at passenger terminals, and some roads have put in experimental tests of various kinds of such construction at outlying points. So far as I know, such construction in tunnels and at passenger terminals is a success. Street railways have been using concrete track support in the paved streets of cities, successfully, eliminating almost all track maintenance.

Permanency in Track.—Since the advent of the automobile, the public road builders, after but short experience with the gravel road, adopted as a standard for high-speed and heavy-tonnage traffic, a hard non-flexible surface. It is believed by the writer that if a smooth, even, non-flexible surface can be provided, that the resilience will no longer be a factor necessary in railroad track construction. With this in mind, we have reached the conclusion that a permanent construction of track is practicable on railroads that have been in use for a period of years, where the banks have reached their final settlement and have had the refinement of line and grade revisions. It was thought that a construction of this type will relieve the high stresses in the rail.

I know of no better way to outline the detail of the form of construction proposed than to quote from the article prepared by Paul Chipman and myself, which was published in the Railway Engineering papers early in December.

By permanent track is meant a track structure in which wear and deterioration are confined as far as possible to the rail. The requirements for such a track structure are most exacting, so much so that in certain situations they cannot all be met with the design here submitted, although it is not unlikely that a type of structure can be developed which would meet even these severe requirements. Such conditions include fills that are not fully settled, fills over sink holes or over ground filled with water

which may be removed subsequently by drainage, soils of such texture that freedom from heaving by frost cannot be secured by thorough drainage, and places where changes in connections with side tracks are frequent. However, on old lines these situations comprise only a small fraction of the total, and their existence need not interfere with the use of permanent track on the remainder, if such use is advantageous.

Such a structure must be wide enough and strong enough to distribute the load over such an area that the bearing power of the subgrade will not be exceeded. Allowance must be made for impact, for lack of uniformity in the support given by the subgrade, and for the present tendency toward heavier loading. Temperature stresses must be taken into consideration; and protection of the concrete under the rail from disintegration due to repeated shock from the passing loads seems desirable, although trial may prove this feature not to be essential.

The method of attaching the rail should permit of its easy placing and removal. Methods and type of construction must be such that the rail, when laid in its prepared place, will have perfect surface and alignment. However, the design should be such as to permit minor adjustment of elevation by shimming, as occasional slight settlement may be looked for even in seasoned fills, and changes in traffic conditions may make a change in the super-elevation of curves desirable. Ease of insulation is another feature that must not be overlooked. As there will be occasional situations where it will be desirable to retain the present type of track, any design should permit an easy and practicable connection of the two types of construction.

Proposed Design.—These features have all been considered in the design submitted herewith. The supporting slab of concrete is 10 ft. wide and 18 in. thick. Assuming the use of 30-ft. rails, the concrete would be cast in sections of that length. The rail rests on the edges of two plates which are embedded in the concrete, and are perforated in order to provide better bond and more bearing on the concrete. These serve a threefold purpose: (1) To distribute the load and impact over a greater area of concrete; (2) to protect the concrete from shock and possible disintegration due to direct contact with the rail; and (3) to insure a setting for the rail which is absolutely true as to line and surface. In addition, they form a guide for striking off the concrete surface in finishing.

Truss Reinforcement Is Employed.—The plates which form the rail seat also form the upper chord of a light truss, the lower chord of which serves as a part of the longitudinal reinforcement. The vertical members of this truss serve to anchor the rail seat to the concrete, and are extended below the lower chord, so that they may rest upon stakes driven accurately to the elevation of the subgrade. This truss would be shop-made, and the bracing and riveting need only be of sufficient strength to prevent distortion while handling. When set in place, these trusses are connected by adjustable tierods spaced 6 ft. apart, and also by brace frames. Four of these brace frames are used for each 39-ft. section. One of the tierods forms the

upper member of the brace frame. This arrangement gives a rigid framework which can be set accurately in place and will remain undisturbed while the concrete is being placed. It will also serve to transmit the loads into the mass of concrete.

Longitudinal reinforcement consists of four ¾-in. bars and four ½-in. bars in addition to the two 1½-in. x 1½-in. x ¼-in. angles which constitute the lower members of the trusses. The ratio of longitudinal reinforcing metal is about 0.37 per cent. Assuming a Cooper E-70 loading concentrated under the axle, with 50 per cent allowance for impact and uniform bearing on the subgrade, the tensile stress in the steel would be about 9,000 lb. per sq. in. and the compressive stress in concrete would be about 200 lb. per sq. in.

Transverse reinforcement consists of ¾-in. square bars spaced 18 in. center to center near the base of the slab, and ½-in. square bars spaced 18 in. center to center, near the top of the slab. Where brace frames and tierods occur, other transverse reinforcement is omitted. Assuming the 70,000 lb. on a pair of drivers to be uniformly distributed over a longitudinal distance of 5 ft., and again assuming an impact allowance of 50 per cent and that the upward pressure of the subgrade is uniformly distributed, we obtain a unit tensile stress of 14,000 lb. for the steel and a unit compressive stress of about 300 lb. for the concrete.

These unit stresses are ample to take care of any increase in future loads, impact and unequal subgrade conditions. On account of the greater length of the section as compared with its width and the consequent opportunity for more variability in the condition of the subgrade, it is thought desirable to use a much lower unit stress for the longitudinal reinforcement than for the transverse. With modern methods of proportioning and making concrete the maximum of economy may be obtained through designing the concrete to meet the actual condition of stresses disclosed through experience in the use of this type of roadbed. It might seem that some economy in design could be attained by not using so thick a slab and using a higher percentage of reinforcement. On account of the impact, however, it is believed that the additional mass obtained by using a greater thickness is well worth its additional cost.

In order to facilitate construction, the concrete slab should be poured continuously, and not in alternate sections. A metal separator would be placed at the end of the section being poured. After the adjoining section had also been poured, this separator would be removed. It is not planned to leave any well-defined joints between the sections, on the assumption that compression due to high temperature would easily be taken care of by the concrete, the thickness of which would prevent any tendency to buckle; and that tensile stresses would cause a cleavage at the end of each section, inasmuch as none of the longitudinal metal is continuous. When these joints develop, they should be filled with tar, as is now done with concrete highways. In construction, the removal of the separator would be delayed until at least a portion of the block ahead of it was poured,

thus giving the block behind it a start toward setting and insuring a cleavage practically along the plane of the separator.

The rail is held by rail-clips, which are attached by bolts to stirrups bolted to the perforated plates and embedded in the concrete. The bolt is placed with the head up and screwed into a nut which is inserted beneath the stirrup through a recess provided for that purpose. This recess is only slightly wider than the nut, thus preventing it from turning. This arrangement tends to lesson the danger of injury to the bolts in case of derailment, and permits their replacement if injury should occur, or the substitution of a longer bolt in case slight shimming is desirable. No angle bars or fish plates are necessary, as the rail seat and rail attachments serve their purpose.

Secondary Consideration.—With such a design, the rail no longer functions as a beam. A wearing surface and a base for bearing and attachment are all that need be provided, so that a much lighter rail may be used than at present. A suggested design for a 60-lb. rail is therefore shown, which it is believed would be fully as effective as a 130-lb. rail under present conditions.

Rail anchors would not be needed. It is possible that creeping of rail could be avoided by keeping the bolts near the center of the rail tighter than those at the end. If this did not prove satisfactory, the rail could be anchored by one or two bolts near its middle point, thus insuring the expansion and contraction of each rail as a separate unit. If it should prove desirable, which does not appear probable, to give a slight cushioning effect and lessen the impact on the concrete, it is believed that a thin layer of oiled felt beneath the rail would be effective. This felt should be treated with some bituminous mixture in order to preserve it. This mixture should not contain too much tar or asphalt, so that it will not become brittle and break up in cold weather. Perhaps a treatment with crude petroleum would be satisfactory. A layer of felt about ⅛ in. thick, compressing to a tickness of 1/16 in. under the tightening of bolts which hold the rail, and further compressing to perhaps one-half this thickness under an engine load, would probably go far toward reducing the shock upon the concrete beneath the rail-seat. It would also lessen the noise. Insulation between any two sections of track could be accomplished by substituting insulating fiber for the felt. This insulation should extend from the rail joint to the end of the concrete block.

Thorough drainage is essential. On fills made of sand no artificial drainage would be necessary, but on other fills it should be taken care of by tile drains just beyond the edges of the slab, with frequent outlets to the side of the embankment. In cuts the drains should be placed deeper and under the cut ditches, in order to prevent ground water from reaching the track. The proper functioning of these drains is a very important matter and they should be of ample size and have good outlet facilities.

Conduits for carrying all wiring, such as telegraph, telephone, block signals and train control, can be placed in the center of the slab or can be placed elsewhere in the section if desired.

It is not likely that such construction as here proposed would be used to any considerable extent on single track, as its cost would not, in general, be justified by the amount of tariffs. Further, to build it on a single track railroad would necessitate the construction of a temporary track to carry traffic around the section of roadbed under improvement. This feature calls for the consideration of precast slabs to be inserted under traffic. This would seem to be entirely feasible, but it is not likely that as uniform a bearing on the subgrade could be had as if the concrete were molded in place, which might result in a slight inequality of settlement of the precast sections.

On double track, however, traffic could be diverted to the track not being improved, without undue expense or serious interruption of traffic. Under such conditions it is probable that the precast section would be more expensive, even taking into account the expense due to traffic interference. It is quite possible, however, that in certain situations this would not be the case and the precast method would be more desirable.

An Estimate of the Cost.—Estimated cost per mile:

Rail and fastenings	$5,310		
Concrete base	28,793		
Total material	$34,103		
Labor	10,858		
Engineering	800		
Diverting traffic	3,000		
Total per mile		$48,761	
Estimated cost per mile of rail and accessories in present track that would be retired		10,791	
Net cost per mile			$37,970

Anticipated Savings.—The advantages that would result from a permanent track may be classified as follows:

1. Reduction in the cost of maintenance-of-way:
   a. Ties eliminated,
   b. Ballast eliminated,
   c. Track labor reduced to rail renewals and track walking,
   d. Probably reduced wear on rails and angle bars.
2. Reduction in the cost of maintenance of equipment.
3. Reduction in train resistance, resulting in:
   a. Less fuel consumed,
   b. Greater tonnage per train, with fewer trains; or higher speed with less train hours.
4. Greater safety, especially at high speeds.

5. More comfort for passengers, due to smoother riding and freedom from dust.
6. Advertising value to the early users.

The last three items have a value that is very real, although so intangible that it is obviously impossible to estimate it in money. This value would be great for roads having a large volume of high speed passenger business. The passenger traffic of today demands the highest speed compatible with safety, and the same is true of certain classes of freight. In general, track conditions are now the controlling factor in limiting speed. With a track in perfect line and surface, and with no soft spots and low joints, this would cease to be the case, and the high speeds required to compete successfully with other forms of transportation could be maintained.

Following is an estimate of the annual saving resulting from the first three of the above items on a track having a freight traffic of 30,000 gross tons per day and six daily passenger trains:

Estimated annual saving per mile of track:

Maintenance-of-way	$1,675
Maintenance of equipment	1,526
Fuel	420
Increased tonnage rating	200
Total saving per mile per year	$3,821

This saving equals 10 per cent of the estimated cost.

A similar estimate, based on a freight traffic of 40,000 gross tons per day shows a saving per year, per mile, of $4,950 or 13 per cent of the estimated cost; and when applied to ruling grades only, of $7,434, or 19.5 per cent.

It is assumed that the cost of maintaining equipment would be reduced 20 per cent. This may be either too high or too low, as there is no basis for an estimate; but it is reasonable to suppose that equipment running over a perfectly smooth track would require less repairs and would have a longer life than if subject to the innumerable shocks inherent in operation over the present type of track.

There is likewise no definite basis for the assumption of a reduction in train resistance of one pound per ton. There is doubtless a large amount of energy lost in shocks at rail joints in the present track structure; and, due to the imperfect elasticity of the roadbed, a further amount is lost in depressing the track and pushing the main track wave ahead of the locomotive and smaller ones ahead of each truck. This loss would be practically eliminated with a track of the type proposed, but the decrease in train resistance which would result can only be determined by experiment.

The conclusions to be drawn are that, for roads with heavy traffic, track of the permanent type would result in a reduction of maintenance-of-

way expenses that would yield a moderate return on the investment cost; would probably further result in a large saving in maintenance of equipment and cost of transportation; and would permit the safe operation of trains at considerably higher speeds than are now permissible.

With moderate return assured on the investment from the saving in maintenance-of-way expense alone, it would be well worth while for any road with heavy traffic to install one or more experimental sections of permanent track in order to determine its other economic values, and to develop by trial the details of a practical design.

It is the intention of the Pere Marquette Ry. Co. to install a short section of this track during the present season. I have no doubt but that the experiment will result in modifications of the design submitted. We hope that the engineers of other railroads will proceed to experiment with some design of permanent track, either the one suggested here or preferably a design of their own. We feel that in this way we will be able to determine the necessity, if any, for placing a resilient substance between the rail and the concrete. If it is determined that resilience in any degree is needed, then it can be accomplished by placing various types of shock absorber beneath the rail and without loss of any of the money expended in the structure.

DISCUSSION.

Mr. Chipman.

PAUL CHIPMAN—In 1894 the freight traffic on railroads in the United States was equivalent to eighty billion tons moved one mile. Thirty years later, in 1924, this traffic had increased to 390 billion ton-miles, or nearly five times the amount in 1894. This is some indication of the tremendous traffic requirements of the future. Freight traffic is now increasing at the rate of approximately 10,000,000 ton-miles per year. It is probable that most of this increase will accrue to the railroads which now have the heaviest traffic.

The weakest factor at the present time in attaining speed with safety is the track. Railroads must develop some type of track structure which will permit, with safety, higher speed and at the same time avoid, insofar as possible, the enormous maintenance expense which is attendant upon the use of high speed on the present traffic structure. The purpose of the design explained in Mr. Alfred's paper is to stimulate interest in this matter, and impress the importance of such a development upon the railroads. There is nothing sacred about this design. The designers have no special pride of creation and it is offered to this society as something to shoot at. We would be pleased to have any comments, favorable or unfavorable in regard to this design from the members of this society.

There are a number of things about a permanent roadbed which can only be learned by experiment. One of these things is the amount of resilient material or cushion, if any, must be placed between the rail and the rigid support. I believe this can only be learned by actual trial. Another thing that will be learned is the feasibility of lifting these blocks to take up any settlement which may occur in the roadbed, and supporting them by blowing or tamping sand under them to the original grade. Some settlement is to be expected even in an old roadbed, and provision must be made for taking up a limited amount of it.

The experimental section which the Pere Marquette intends building will enable some idea to be formed of the decrease in train resistance which may be expected on a track of this type, and some idea, but not very much, as to the effect of the maintenance of the equipment. It will give an opportunity of trying out different kinds of railway attachments to find whether or not it is necessary to have angle bars and fish plates. One point, which was brought out by Mr. Grandy some time ago, was the advantage of construction of this type in doing away with all beam action in the rail. The rail is not subject to any tensile stress, and may be made considerably harder than with the present type of track. This would probably result in a considerable reduction of rail wear and a consequent saving in maintenance.

The Railroads and Concrete.

A Discussion.

T. P. Watson (*By Letter*).—It has been asked: "Are the railroads progressive in reference to concrete design and construction practice, and if they do not appear to be, what special conditions apply in railway construction work that are the basis of the railway engineer's viewpoint?" **Mr. Watson.**

In answer to the first part of this question, the railroads are not only progressive but they were pioneers in the use of concrete for structures carrying excessively heavy moving loads.

Railroad engineers designed and are continuing to design new types of concrete structures that are subject to shocks and exposed to weather conditions not encountered in other fields of concrete construction.

As to general practice, the construction engineering forces of our railroads are second to none in ability, and the comparative excellence of the vast volume of work accomplished is proof of this statement.

An attempt to answer the latter part of the question: "What special conditions apply in railway construction work that are the basis of the railway engineer's viewpoint?", permits the calling of attention to only one phase of concrete construction on railroads. Covering as they do such a tremendous geographical area and possessing varying financial conditions and prospects for the future, a full discussion of this question would take the time of many conventions.

Many railroad engineers, like engineers in other fields, had assumed, because of custom and the erroneous idea which developed in the past twenty-five years, that concrete was a fool-proof material so long as tested cement was used with arbitrary proportions of aggregates, regardless of grading, mixed together with an entirely neglected proportion of water and was *poured* into a form of the proper dimensions with casual if any supervision and inspection.

The execution of a considerable volume of railroad concrete work is placed under the charge of men already busily occupied with urgent duties, so that the actual time they can devote to field supervision of concrete construction is negligible. The duties of these men involve the installation of a vast assortment of manufactured materials and appliances which, before their receipt, almost without exception have been carefully manufactured under rigid specifications, inspected and certified to by competent inspection organizations.

It is something of a mystery, but true, that concrete seems to have been accepted on the same basis, whereas the making of concrete is a manufacturing process and should be recognized as such and that special knowledge and skill are required to obtain the best quality of work.

As an example of the methods and extremes of the use of two con-

struction materials under such supervision let us consider the building of a steel girder bridge with concrete abutments carrying a railroad over a highway. The work is authorized, a plan prepared and the concrete abutments built and completed and the only inspection or test made in connection with the concrete is probably that made of the cement before it leaves the cement mill.

The steel for the girders of this bridge is an entirely different matter. Tests and inspection start on the material that enters this part of the structure from the time the ore leaves the mines, through all the transition periods of this material, until the last field coat of paint has been applied.

How is this laxity in the manufacturing of concrete to be overcome? Some will think that a rigid specification is the answer.

Specifications for concrete work on railroads of any considerable length should only be made after a careful consideration of the economic phases of the availability of aggregates, as the perfect aggregate of one section of the road might be an economic absurdity at another point. Aggregates must be analyzed from each source of supply and the concrete specifications drawn accordingly.

Even a perfect specification is practically useless unless competent inspectors are present at all times during the progress of the work. The inspectors themselves must be under supervision which will keep in close touch with the operation to *personally* observe if the inspector is competent and co-operate with the contractor or construction forces to obtain finished work of the best quality. This comment on how it should be done will bring forth the inquiry, "How are we going to get the men to carry out such a beautiful theoretical plan?"

A little study of the recent research and discussion of concrete is very enlightening and it is a simple matter in most localities to find concrete structures that have failed or are deteriorating to such an extent that their renewal is only a matter of a short time. Intelligent study of these failures will prove almost without exception that the laxity of inspection and supervision was responsible for the poor results obtained.

With these examples as a basis, it should be a comparatively easy matter for railroad engineers to impress upon their superiors that the necessity for special men for the special work of field control of concrete is just as essential as the specialization which is taking place in every other field of modern business activity.

Further make an estimated cost of proper inspection and supervision and compare this cost with the renewal of a concrete structure at a period of twenty-five years with the renewal at a thirty-five-year period, on the conservative assumption that on the average you may increase the life of a structure ten years by proper inspection and supervision. If you will make your estimated engineering cost higher than is reasonable, the net results with compound interest are a revelation.

Railroad managers are high-class business men and economists and

if the truth of the economy of adequate inspection and supervision of concrete construction was brought to their attention in the proper light these wasteful conditions which exist on some roads would cease immediately, as they did long ago on many railroads.

Mr. Haggander.

G. A. HAGGANDER (*By Letter*).—The C. B. & Q. R. R. was one of the earliest large users of reinforced concrete in railroad bridge construction. This was brought about by the confidence of C. H. Cartlidge in this material and he developed the use of concrete box culverts, trestles, deck slabs for steel bridges and other types of construction to a great extent.

Good rock for concrete aggregate is not available on most parts of our railroad but there is an ample supply of gravel. Until last year practically all of our concrete was made with bank run gravel from three sources. These materials all contain a large percentage of sand and required the use of larger amounts of cement than usual, this amount varying from 1 to 3 sacks per cubic yard over the usual practice.

In 1924 one pit was worked out and we began using separate aggregates on the Eastern portion of our line, thinking that the cost of concrete would not be increased due to reduction in amount of cement and that a more uniform product would be obtained. We specified the usual 1: 2: 4 and 1: 3: 6 mixtures but soon found that the resulting concrete was very difficult to work and that the voids in the stone were not always filled. We have therefore had to use extra sand. Our trouble was partly due to not taking into account the bulking of damp sand. Having in mind the issuance of instructions covering the proper proportions to be used with the materials available, we have had our materials analyzed and tested at the laboratories of the Portland Cement Association during recent months and have developed proper proportions to use for each of our three classes of material.

We find that we were using about the right amount of cement with our pit-run gravel but that for the separate aggregates, the volume of sand required under field conditions is as great, and for some strengths greater, than the volume of stone.

We began to realize in a vague way in 1917 that too much water was being used in some of our concrete work. Instructions were issued at that time to make concrete only soft enough to work readily but not so wet as to leave water standing on it after being placed.

We did not understand the relation of water to the strength of concrete but desired to limit the amount so that the cement would not be washed out or the aggregate segregated in handling.

In order to check up the laboratory tests we have used the new proportions on a job now partly completed.

We desired a concrete having a minimum strength of 2,000 lb. and average strength of 2,500 lb. at 28 days. Field tests showed a minimum of 2,520 lb. and a maximum of 4,000 lb. with an average of 3,160 lb. Our measuring of materials in each batch was not accurate and about ¼ sack

of cement per cubic yard excess was used as shown by a final check on completed sections. This accounts in part for the excess strength.

This is our first attempt along the lines of proportioning concrete by the more recent theories, and the results are satisfactory. The excess cement used with pit-run gravels has insured a good quality of concrete in the past but we were concerned about the results we were getting with the standard mixes when using separate aggregates.

As we use only a limited number of kinds of aggregates, it is our intention to apply the information obtained to proportioning concrete for small jobs arbitrarily without further tests and to make field analyses on larger jobs only.

In case these show considerable variation from expected results we will have to extend the field analyses to smaller jobs.

Our instructions will be very strict as to the water cement ratio which seems to be the main factor governing the strength of concrete. We feel that the foremen on our small jobs can be made to realize its importance and will impress on them that in case our specified proportions do not give a workable concrete, both cement and water must be added to remedy the condition, instead of water only. This would not be economical on a large job but I feel that is the best way of handling small jobs having no inspector.

Mr. Westfall. MR. WESTFALL.—I have heard the question asked whether the railroad engineer was progressive, and if not, why, and I came here to find whether those questions would be answered. I think the situation of the railroad engineer is somewhat different from that of engineers in other lines, when his work is applied to traffic structures. Safety is the ultimate and the big question, and he has to be pretty sure before he does things that those things are going to be right. He is therefore inclined to be quite conservative and to try to be, as far as possible, right in his assumptions. There is some question possibly as to whether the railroads have taken up the latest ideas of the manufacture of concrete as rapidly as some other branches of builders, but I believe that in types of construction the railroads are not far behind.

Speaking of unit or precast construction, it seems to me that the railroads have probably gone as far in important work as any other class of engineers. In track slabs, the use of pre-cast piling, platform slabs and concrete crib work, there are almost innumerable examples of the application of this construction.

There is one point of considerable importance in concrete work and that is the matter of design of construction detail. I think that in concrete failures there have been many more instances where the failures were due to the overlooking of comparatively small details rather than design. I believe that the railroad engineer is in a more enviable position to watch his concrete work than the average engineer in outside practice. In most cases he handles the design of the structure, he looks after the construction, and then he has to carry the load afterwards; if anything goes

wrong, he knows about it. Unfortunately in concrete work it is very difficult and probably impossible to tell what is the matter. There are so many things that can enter into concrete work that can make it go wrong.

In regard to one point that was brought out by Mr. Watson, which I think rather hits the nail on the head, that people took up concrete with the idea that it was fool-proof, I think the reason was that concrete was first taken up to replace gravity masonry. In the old days cut stone work was really brought up almost to perfection, but in railroad structures, piers and abutments, we find that a great many of these structures were built with very nice cut stone faces, and the insides filled with spalls and mortar. Sometimes they were comparatively dense and very often they were rather full of voids; so that concrete was really developed on railroads to take the place of that kind of work. Possibly that is a justification of the attitude that the early railroad men had in reference to concrete, that it was equal to or should have been equal to the masonry work it was replacing.

Abutments and piers do not have to be as good, considering concrete, as the concrete work in reinforced slabs and columns, so that idea has to be lived down by the railroad men.

A. BURTON COHEN.—I think the trouble with the engineering departments of the railroads is that they are curtailed too much by the management. I have had considerable experience in railroad design and construction work, and I find that the tendency now is to cut down everywhere the railroad's engineering department forces. The help which the bridge engineer of the railroad is receiving is of an inferior quality. The management takes the point of view that the engineering department is spending money. The management also pays more attention to the departments that are bringing in the money by traffic than to the engineering department. We speak of failures in bridge construction. Why do we have these failures? It is because railroad officials expect an inspector of insufficient experience who is receiving a very small salary, to take that responsibility. The grade of assistance that must be given to the bridge engineer must be improved if the work of the railroad, the very good work that has been done, is to continue. **Mr. Cohen.**

ROBERT H. FORD*—The main thing I assume in which you are interested is why, if the savings indicated can be secured, the railway engineers are so short sighted that they are unable to practice greater economy. I can visualize the taxpayer sitting in here in a corner of this meeting and going away with the feeling: "How they are burning up my money." I am going to emphasize what I might say before our own convention, the American Railway Engineering Association. There is a woeful lack of practical application of the theories which such conventions as this produce. It seems to me that what associations of this character need is **Mr. Ford.**

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some sort of an outstanding committee on practical research. Why is it, with so much of this excellent material developed, that it is not practical or at least that it is not used? Why is it that we do not apply the things that you men write about and develop? If one-quarter of what was said here today was true, and I have no reason to doubt it, somebody, some organization, whether railway, highway or otherwise, is spending rivers of the people's money improperly and unwisely.

The answer is that men like you do not seem to be able to get the thing over to the public. So I say that one of the first and most important things for an organization of this kind is an outstanding committee on practical research.

Turning to the railroads. you are interested primarily in the use of concrete. Transportation is the most important industry in America, after agriculture. It is divided today into about three grand divisions, the railroads, the waterways and the highways. The highways today have jumped into first place, I think, as the users of concrete. The time is going to come, however, when, I believe, the railroad will take that position, but it will be a long time. Railroads are only about a hundred years old. As a matter of fact, modern railroading is only about twenty-five years old. We have passed through three crucial stages, and it is only within the last few years that the public is beginning to appreciate what transportation means. It is the life blood of the nation. Certainly, if it is as important as I say, it should be built well and wisely. Of all the mileage in this country, only about 20 per cent is what we can call permanently located. The rest is yet to be reconstructed and built. A single track railway is only half a railway. The grades must be reduced, the curve must be eliminated or lessened and a great deal of reconstruction must take place before we are up to the point where we can say that we are in a position to build with any degree of permanence.

The discussion today dwelt largely on bridges. Bridges constitute only a small fraction of what we can call the right-of-way. I wonder sometimes in this practical question of research, why you men who are connected with the output of concrete do not study somewhat the development of locomotives or the cars and their sale and manufacture and development. The story is simple. It is a plain question of finance. The railroads cannot finance permanent work. A second-track railroad, modernized with the grades reduced, curves eliminated or lessened, so that one could travel a hundred miles an hour would require 35 per cent to 50 per cent of the present investment in railroads. What is the use of preaching about concrete for permanency? What is the use of spending your time, so far as the railroads are concerned, with problems that are purely academic?

You men are practical men. The majority of you can look over the top and see the practical side of things, and there is only one word that spells it today in modern industry, and that is finance. The problem today of transportation is finance. When railroads cannot make a fair return on the investment which has been set up by the Interstate Commerce Commission, which is nothing more than a medium which represents you men, it

is idle to talk about proper, efficient construction. I will say that, as soon as the money is forthcoming, as soon as you men can find some means whereby you can get classification of account, so that the railroad can borrow money on short-term notes for permanent work in the same manner as they can borrow it for locomotives and cars, we will make far more transportation industry progress. We will use more cement, we will have better concrete and it will cost you less to carry your goods.

Earthquake Proof Construction.

By H. M. Hadley.*

The prerequisite of any problem of engineering design is a knowledge of loads and forces involved, and to this rule the problem of rendering buildings and other structures earthquake proof is no exception. Before any beginning can be made on the problem something must be known of the earthquake itself, what it is, and what is to be expected at times of earthquake occurrence.

Briefly, an earthquake consists of sudden and always unexpected movements in what all our normal experience leads us to regard as the fixed, immovable earth. These movements may be so slight as to be almost imperceptible; again they may be so violent as to wreck all but the strongest structures. They may be brief; they may be prolonged. They may occur anywhere, at any hour; they may not occur for decades, or even centuries. It is doubtful if with present knowledge they can be predicted in any save the most general way. Certain zones and districts are known to be more subject to earthquakes than others, and from a study of past performances and correlated phenomena, some fore-knowledge may be obtained, but from the very character of the earthquake, precision is scarcely to be hoped for.

The earthquake movements are both vertical and horizontal, the horizontal being in general six to ten times greater than the vertical. Since all structures are designed for vertical loads and presumably with an ample factor of reserve strength, the stresses induced by the vertical component of earthquake motion may be ignored. No ordinary, well-built structure with good firm foundations will sustain any serious damage from such vertical movement.

But the horizontal components of the earth movement may in no wise be ignored. Depending on the magnitude of the horizontal movements and the time in which they occur, forces of varying magnitude are developed and in cases of severe earthquakes these horizontal forces will destroy or severely damage all structures lacking in resistance to them. At any locality this horizontal earthquake movement may occur in any direction, consequently it is necessary to provide resistance against earthquake forces acting in any direction. A severe earthquake may easily develop a horizontal force applied to the base of structures equal to 1/10 or more of their combined live- and dead-loads. However, if actual designed provision, employing the usual factors of safety, had been made and executed against

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horizontal forces of 1/15 or even 1/20 of the total loads, it can safely be said that the great majority of the damage done by the Santa Barbara and Tokio-Yokohama earthquakes would not have occurred.

When foundations and column bases are suddenly moved sideways by the irresistible movement of the earth, there must be strength in the vertical members of the structure above either to bend and deform with

FIG. 1.—TWO CONCRETE SCHOOLS AT SANTA BARBARA.

The Santa Barbara High School at the top had well designed concrete wall construction and sustained no damage. The Wilson School (below) was of skeleton construction with clay tile filler walls and was seriously damaged.

adequate resistance to the stresses produced by such bending and column eccentricities, or to carry the structure bodily as a block or unit with the foundation and without major deformations. Economy, and the character and use of the structure, will largely determine the type of resistance to be provided. It is pertinent to call attention to the fact that with the possible exception of metal lath and plaster, none of the finishing materials employed in buildings of any consequence is flexible. The essential incon-

gruity between a flexible structural frame, and these inflexible things which the structural frame must sustain has resulted in many a sorry spectacle—and the loss of millions of dollars.

For the great majority of buildings and structures it will be found that the stiff-frame type is the most practical, adaptable and economical for resistance to earthquakes. Once it is decided that provision is to be made

FIG. 2.—TWO WOODEN BUILDINGS AT SANTA BARBARA.

The gymnasium at the Teacher's College (above) possessed well designed, well-braced wood frame. The stuccoed exterior was not even cracked. The two-story residence shown below had no bracings on its underpinning.

against horizontal earthquake forces the magnitude of these forces must be assumed as some fraction of the vertical loads. Considering the fortunate infrequency of earthquakes at any particular locality, reliance may well be placed on the reserve strength provided by the usual working stresses used for normal loads. Consequently, with a safety factor of 3, 1/15 may be recommended as the coefficient giving an ultimate resistance to an earthquake whose dynamic effect is that of a force of 1/5 of the total weight.

Earthquakes more severe than this are very rare. Having selected some coefficient—say 1/15—then in each story design must be made to resist horizontal forces equal to 1/15 of all vertical loads in columns and walls in that story. These forces are analogous to wind pressure and the design against them is similar to the design against wind loads. They measure the shearing force in each story and the tendency of the portion of the building below to move horizontally with respect to the portion of the building above.

Resistance to these forces can be obtained in the stiff-frame type of building either by trussing or by a sufficient amount of exterior and interior reinforced-concrete wall construction. Such wall sections are figured as beams, for bending, shear, bond, etc. The reinforced-concrete wall is

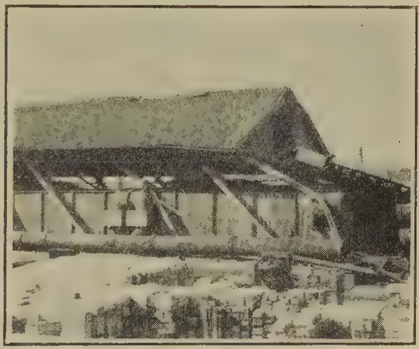

FIG. 3.—TWO STEEL-FRAMED BUILDINGS IN JAPAN.

The seven-story building of Katakura & Co., Tokio, shown on the left, had concrete walls and sustained no earthquake damage. The one-story gas purifier shed of the Yokohama Gas Co., shown at right, collapsed on account of lack of adequate bracing.

alone recommended simply because it is the only wall in which it is practical to introduce reinforcement to resist the tensile stresses that occur equally and simultaneously with the compressive. With either diagonal bracing or structural wall, a good distribution with respect to the plan of the building is important: it should not all be localized at one point. The corners of buildings constitute especially effective points for such sections, and there is the further advantage that the sacrifice of light there is a minimum. Stair and elevator enclosing walls may also well be used.

When adequate structural resistance has been provided, the remainder of the wall and partition construction may be of any of the common masonry types, or of glass, or may be left open to the air. But full skeleton construction, so common in American practices today, is certainly financially, and quite possibly structurally, unsafe at times of severe earthquakes. With the exception of certain steel-framed mill buildings covered

with galvanized iron, neither at Santa Barbara nor in Japan did the speaker see skeleton-framed buildings of any material that escaped damage. Hope is a poor material for bracing.

FIG. 4.—TWO SMALL BRICK SCHOOL BUILDINGS AT SANTA BARBARA.

These buildings are of the same size and stood on adjacent corners of the same block. Observe difference in wall construction. The small buildi[illegible]

Whatever the materials of construction, every effort should be made in the design to tie and interconnect as fully and adequately as possible, all structural parts, and to attach firmly to the structural frame all filler and decorative material used for the completed structure.

DISCUSSION.

A. G. HILLBERG* (BY LETTER).—Before starting to analyze a design and its sufficiency to withstand successfully the loads it might be called upon to sustain, we must, first of all, know the nature of these loads and their application on the structure. Live- and dead-loads can usually be determined by using unit loads and unit weights contained in building codes and hand books. Their action is well known. Mr. Hillberg.

But, when starting to analyze *earthquake stresses* in structures, we are on the wrong track—because there are no such stresses. This statement needs explanation as it sounds unbalanced in view of the damage done from time to time by earthquakes. Suppose a ball of fragile material were rolled along a smooth floor and it met an unyielding obstruction, for instance a masonry wall, and it broke, what agency would be directly responsible for its destruction? Of course the inertia of the ball itself, that is, its tendency to continue in motion. It is quite true that indirectly the setting in motion of the ball is originally responsible, but not directly so, as it happened at an earlier period of time and its direct effort or power imput on the ball had been concluded. Therefore, when analyzing a structure as to its resistance to stresses produced in it by earthquakes, this must be borne in mind and the inertia of the structure itself used as a basis rather than the earthquake shock itself.

We all know the trick of pulling a cloth off a table set with the usual china and glass service, removing it by one quick pull and leaving the service on the bare table in the same position it had on the cloth. Why did it not follow the table cloth and fall off the table? Because of the inertia, or tendency of each piece to remain at rest, and the failure of the rapid pull to transmit itself to the pieces with sufficient strength to overcome this inertia. But had some of the pieces been stuck to the table cloth, glued for instance, they would have followed the cloth. This bears out a great principle in the design of earthquake proof structures:

Do not anchor them; cut them off from any subsurface structure as near the ground as possible. A conclusive proof of this is shown by the fact that structures most damaged are those on pile foundations, those having deep basements and those founded on rock.

But it is not always possible to do away with basements, nor is the ground always solid enough to permit spread footings. True, but in such cases finish off the basement at or near the ground level and start on it with a new foundation for the superstructure. Therefore, the first rule will be:

Separate the subsurface work from the superstructure.

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However, the superstructure of a large building is heavy, and the friction coefficient between the two surfaces in contact at the ground level is so great that a considerable force will be transmitted, and this introduces the second rule:

Provide an elastic layer between the subsurface work and the superstructure.

The best insurance would be to introduce roller bearings, similar to those used for steel bridges, but a layer of pit-run gravel will do, provided it is so confined that it will not run out. In concrete buildings such a layer must be covered by a built-up composition roofing before any concrete is cast upon it, so as to prevent grouting the layer. Even with these precautions, it is impossible to eliminate entirely the transmission to the superstructure through friction of part of the horizontal force acting on the substructure, but same can be reduced in proportion to the elasticity or flexibility of the joint.

In the design of the substructure several important points must be observed such as the tying together of all foundation piers, the bracing of foundation walls, and in all possible ways insuring the fact that the different parts will move and act as a unit. This is easily done through the introduction of heavy reinforced-concrete walls so designed as to produce a cellular structure of great strength. In places where earthquakes of great severity occur and the disturbances in the crust of the earth are considerable, I believe it to be advisable to secure relief by building the foundation walls in the form of heavy piers, with thin slabs in between; these slabs to be so designed as to break and to permit a cavein of the ground, thus relieving the direct pressure on such walls. This will not lessen the energy imput on the structure, but will safeguard against collapse of the subsurface work.

It is impossible to eliminate entirely the transmission of at least a fraction of the force of the superstructure. In order to understand better the stresses to be expected in a structure due to this force, we shall use as an example a building with basement placed on a pile foundation and with sub- and superstructures rigidly connected. We shall also assume that the building is of good design and built as nearly monolithic as possible.

We also know that an earthquake tremor is not like a wave action on a rough sea, but rather like an alternating electric current, that is, it has an oscillating movement. During the first period this movement is forward, i. e., away from the earthquake center or origin. Passing through the earth it encounters the pile foundation and the other parts of the substructure, moving them along with it. Due to its inertia the superstructure is unwilling to be set in motion and resists until a force large enough has been created to make it move. It starts its forward movement and it is an accelerated motion gaining in momentum.

Then the second period starts and the direction of the earth movement is reversed and with it the foundation and the basement of the building.

But the superstructure, due to its inertia, tries to keep up its forward movement and continues to do so until a large enough force has been produced by the earth movement to absorb its energy and to bring it to a standstill, when the superstructure also starts a reverse movement. Due to this "lag" in movement a "fanning" action is produced and the sway of the building becomes greater and greater until it collapses or is damaged sufficiently to become more or less elastic at a certain level.

From the above it is obvious that the stresses such a building is subjected to are far greater than the actual earthquake force acting on its substructure, and is also a function of the duration of the tremors. It is also evident that the inertia of the building and the location of the center of its mass are important factors in the determination of the stresses it is subjected to. If this were not the case the stresses produced in all structures, large or small, would be the same inasmuch as the earthquake force is the same for all of them. This is also borne out by the fact that low, spread-out structures suffer less than tall and narrow ones. Obviously the third rule becomes:

Place the center of the mass of the building at as low a point as possible.

As the mass is the weight divided by g (32.16), this rule can be interpreted to mean making the lower parts of a building heavier than required by the dead- and live-loads above and also using lighter materials in the upper portions than in the lower ones. But I believe in a much more effective way than that, one that will not be readily approved by the building departments of this country, but which has already been used in the design of the Imperial Hotel in Tokio, one of the few buildings to come unscathed through the recent earthquake. We all know that the live-load requirements of the customary building codes are far heavier than they need be. In a hotel room, for instance, a live-load of 40 lb. per sq. ft. is stipulated and all that room will ever contain is a bed, a dresser, a few chairs, a table, a trunk or two and one inhabitant and his guests. As guests are not usually received in bedrooms, it is a fair assumption to limit the total number of occupants to four, or in all a live-load of 10 lb. per sq. ft. in a room about 10 x 12 ft. in plan.

Actual live-load conditions in office buildings and apartments are about in the same proportion. Furthermore, the higher a building, the less the chance of heavy loads on its upper floors. Therefore, lessen the live-loads on the upper floors, using a sliding scale.

Permissible unit stresses in building materials are also such that they can be increased materially without endangering a well-built structure; and they should also be changed in accordance with a sliding scale.

These changes will result in a building with its center of mass as low as possible, without having to use wasteful methods in providing material only for the purpose of making heavy the lower portions. Furthermore, the use of such extra material should be avoided on the ground that they increase the mass and consequently the inertia, thus increasing the stresses.

Obviously, heavy roofs should be barred as they place a heavy load at the top of a building. In Japan and in the old Spanish type of houses in the Philippines, heavy roofs have caused much damage by being supported on frame structures, entirely too weak to hold them after they once have been set in motion.

So far nothing has been said as to the magnitude of the earthquake force, and this is a subject on which it is impossible to generalize. Each earthquake center has its peculiarities with respect to geological formation, location of faults in the earth structure, depth at which the shock is transmitted, etc. The only correct method is to get all available earthquake data from the nearest observatory and from the accelerations and amplitudes compute the maximum force on record and then, if thought advisable, increase same to cover possible conditions of a severer nature.

Any fault lines in the formation should be carefully determined, as settlements are most likely to occur along these. No structure, of whatever nature, should ever be permitted to straddle such a fault. I know of one city in an earthquake center, where the main water supply pipe from the reservoir to the city crosses such a fault. Should a severe earthquake occur the city water supply would be cut off.

This opens up a closely allied subject and one based on the fact that most of the damage done in a city is not so much due to the earthquake itself as to the numerous fires it creates. Modern cities have electric conductors, gas mains, gasoline filling stations and numerous automobiles and trucks with gasoline in their tanks. Earthquake-proof construction, therefore, goes hand in hand with fireproofing and fire prevention and in order to make a community more or less earthquake-proof, these details should not be overlooked.

For instance, a gas main should never be run directly into a building and cemented into any subsurface wall. It should be looped both outside and inside the building thus leaving opportunity for expansion, contraction and distortion of the ground without breaking it. It should run into the building through a large hole packed with elastic materials and flashed with more or less flexible membrances on both sides of the wall.

After the probable movements of a building have been determined and the forces acting on it calculated by using the inertia of the entire superstructure, these forces must be applied on the structure in proportion to the distribution of its inertia. Here it must be observed that the upper portions will have larger gyrations than the lower ones so that this distribution will by no means be uniform, as in the case, for example, with wind loads.

To design a structure which will have a fair amount of resistance to stresses produced in it by earthquakes is by no means difficult. As to the material most to be preferred, nothing can compare in strength with well-built reinforced concrete. During my many years of residence in Manila, P. I., it has been a privilege to design and build a number of structures and to try out different theories. My only regret is that during my stay

the not infrequent earthquakes experienced were of minor importance and no damage could be detected in these buildings. In the designs have been incorporated a flat-slab floor system in a two-story building, a combination of structural steel with cast walls and floors in a three-story building, a five-story office building with beam-and-girder floor systems, columns designed to resist lateral deformations, columns of naked structural steel with heavy outside concrete walls, floors with diagonal ties at corners to make the wall lintels work as fixed horizontal beams to distribute shocks attacking the building diagonally, reinforcing steel laid diagonally both ways in all floor slabs, floating foundations, on rock with gravel fill under, such on medium and soft ground, deep pile foundations well tied together, interior walls of light materials having no structural strength, interior walls of concrete cast monolithically with the columns and outer walls, etc.

Some day a lot of interesting material on the action of a severe earthquake on these buildings will be available, and I hope to be able to make the examination personally and with the knowledge of the assumptions which served as a basis for the design, draw the conclusions.

EARTHQUAKES AND THEIR EFFECT ON BUILDINGS.

BY ARTHUR L. DAY.*

Before I may talk with you about building for protection against earthquakes it is necessary for me to explain that I am neither architect nor builder; that I probably shall miss many features of technical description which you might be led to expect from me; but that I have had a number of years of experience of the study of earth movements, and it is of these that I should like to speak.

The earth is an ancient edifice a few hundred million years old, and it is generally agreed that at one time the surface was hot. It is also a fact of common observation that the materials which go to make up that part of the earth with which we are acquainted are very heterogeneous. The rocks are of many kinds with different elastic constants, and different expansion coefficients; they have been cooling to their present condition through a long period of time, so long that probably the important stages of the cooling operation have pretty well escaped our observation. Nevertheless, we are dealing with a structure, and all engineers will speak the same language in considering a structure as heterogeneous as the earth; they will have the same expectations of weak spots in such a structure. the adjustments of which, as it grows older require some attention if we are to live safely upon it, because such adjustments will inevitably result in strains and ruptures here and there. We recognize mountain ranges as the result of some such operation. Most of the mountain structures that we know were built long before our time so that the operation of mountain building is not accessible to us. Nevertheless, there are adjustments going on within this earth structure today in the same fashion in which they were going on a few million years ago, although probably of very much lower intensity and smaller magnitude now than then.

It is entirely possible to study the earth in the same way that you would study any other great structure. You can tell where the weaknesses are, partly by looking for cracks, which you will find in plenty, partly from the differences in the elastic behavior of its materials when an earthquake disturbs them, and partly from the wide differences of temperature near the volcanoes and those regions which are still actively in process of alteration. Under all such conditions the weaknesses of the earth structure become accessible and our attention is properly and normally directed to them.

Geologically, the west coast is new and the east coast is old. In terms of earthquakes the west coast has felt a severe shock every twenty-five years or so since history began to be written in that region; the east

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coast has had no more than three such shocks in the 250-odd years for which records exist. Moreover, the earthquakes of the east are usually of lower intensity than those of the west. For these reasons the west coast has attracted the attention of students of earth movements. The report of the California Earthquake Commission (Prof. A. C. Lawson, chairman) published in 1908 represents a most careful study of the great earthquake of 1906. A year ago the Seismological Society of America published a fault map of California. That fault map gave a surface representation of the State of California with the structural cracks or faults, so far as they are known, marked upon it.

The geologists have also recorded the fact that California has a curious geological history. They tell us that the Sierra Nevada Mountains, some 14,000 ft. high, have moved vertically no more than 1,000 ft. during their recognized geologic history. They tell us also that the Coast Range Mountains have at times been as much as a mile below the level of the sea and at other times have reappeared somewhat as they are now, and that four or five of these up-and-down movements have probably occurred. Those of you who read the report of the California Earthquake Commission after the earthquake of 1906 will recall the statement made by Professor Lawson, with the full authority of all the geologists collaborating with him, that the west coast of California had behaved something like a trapdoor, moving about the Sierra Nevada Mountains as a hinge, and that this was readily proven by the fact that marine fossils were found which could only be there as a result of exposure to the sea, and alternating with these were found layers of sand and clay, which could be deposited only by the erosion of nearby mountains. If we have a structure showing so much evidence of movement as the west coast does, it is plain that we have a right to expect and to look for signs of the structural weaknesses associated with these movements of the past, which, fortunately, are not so violent now as then.

When the Seismological Society published its map of California with the faults indicated upon it a great many of these cracks were pointed out, the longest of which, the San Andreas fault, extends from near Cape Mendocino in the northwest to the Mexican boundary, something over 600 miles. All of these cracks are substantially parallel to the Sierra Nevada Mountains on the inside and the continental shelf on the outside, which, thirty or forty miles beyond the coast line, drops down some 12,000 ft. along a line which also is nearly straight and parallel to the Sierra Nevada hinge, leaving more than 25,000 ft. (5 miles) between the bottom of that shelf at sea and the top of the Sierra Nevada Mountains. Considered as a structure you will see from this that the state is badly loaded. Inside is the great mountain range with many parallel cracks between it and the coast line, while beyond the coast line the contour breaks sharply a little way beyond the present shore.

From the written history of California we learn details of the individual movements in the southern part of the state. There is a record

among the Spanish missionaries of an earthquake of great severity as far back as 1769. There was another great earthquake in the southern part of the state in 1812, when two of the fine old Spanish missions of California, distant as much as 175 miles apart, were destroyed at the same time. In 1857 there was an earthquake in the region of Santa Barbara itself, which destroyed the Santa Barbara mission and other structures in the southern part of the state. Fortunately, during that early period, there were very few inhabitants in these regions. In the northern part of the state, recalling to mind only the later earthquakes, there was the Haywards fault earthquake east of San Francisco in 1868, and the destructive "San Francisco earthquake" along the San Andreas fault in 1906. It is characteristic of a structure broken by strain that when the strains which cause the break are relaxed time must be given for the accumulation of more strain before further movement is likely. Therefore, inasmuch as we are dealing with elastic materials (rocks) of great heterogeneity, of great differences in temperature, in a region which already has been strained beyond the breaking point at many locations, we have to expect further, more or less periodic displacements.

It is not difficult to say to you as engineers that the points of greatest weakness can now be pointed out and so designated. Then it is not going to be very difficult to tell where the next earthquake is likely to occur. From our present knowledge no one will attempt to tell when it is likely to occur any more than you could tell from a man-built structure, which was known to be dangerous, just when it would fall under the forces which it should commonly resist. So it probably will be a long time before the occurrence of earthquakes will be predicted, but it is not very far to the point when we can predict with reasonable accuracy where they will occur. The prediction of earthquakes in place fixes upon us definitely the responsibility of seeing to it that buildings on those spots are so constructed as to sustain the shocks which are liable to come to them.

If one of your number had spent years in building apartment houses on the granite hills of New England and were suddenly to transplant his energies to Florida where no rock is to be found, I dare say he would not hesitate a minute in revising his scheme of building so as to provide the kind of foundation most appropriate for the Florida location. But I doubt whether the same individual upon moving from Florida to California would ever stop to ask whether the site of a proposed building happened to be near a known earthquake fault. That is the present day attitude toward the earthquake danger. We are at present almost criminally careless about the earthquake risk, both the character and the amount of it. It is possible to appraise this risk to a certain extent. We know the kind of thrust that an earthquake brings. The Seismological Society has pointed out the faults in California from which the elastic vibrations are likely to take their origin. Vibrations, we know, are transmitted longitudinally and transversely. If the fault be located and one knows that vibrations proceeding from it will be oriented in two directions, one a perpendicular

thrust and the other a transverse vibration, one has a certain amount of information of immediate possible application in the building of near-by structures. We know also the amount of displacement recorded during the several earthquakes of which California bears the record. There is therefore nothing but psychological inertia behind the failure to recognize these conditions and the will to meet them to the extent of existing knowledge.

Nevertheless, the situation does not always work out so simply. In 1798, Spallanzani, an Italian traveler visiting Messina, which had been completely destroyed in 1783, wrote down as good a rule for earthquake-proof structures as I have ever read. He said that in order to meet the horizontal thrust directed at the base of a structure, it was necessary either to provide that the entire structure move as a unit under the thrust, or that it have elasticity enough to carry the thrust from its own center of gravity when the shock came. He advised against building structures more than two stories high (at that time of course the steel structures of today were not known). The brick buildings of that day and those of stone above two stories in height he knew would be dangerous in the severe earthquakes that visit the Calabrian Straits. He also recognized the increased danger to all structures erected upon the sandy shore. He then gave plain warning to those who should rebuild the city of Messina that certain building rules must be observed. The city of Messina has since been rebuilt and again destroyed, and it is now being rebuilt for the third time. Without wishing to speak too positively of a situation which I have not seen within the last five or six years, I venture to say that the structures of the third rebuilding will differ little from those which were in the mind of Spallanzani and against which he warned.

I was in Santa Barbara on the day after the earthquake of last summer, and, with Professor Willis, was summoned by telegraph to San Francisco to speak before the Chamber of Commerce of that city, which was considerably disturbed by this severe earthquake which had just visited the coast. The room in which the meeting was held was packed to the doors and everyone was tremendously interested. It was possible in that atmosphere to speak quite plainly, and we spoke quite plainly. The damage at Santa Barbara was not so much due to the violence of the earth movements as it was to poor building. The earth movement recorded at Santa Barbara was small compared with certain of the great earthquakes, notably that of San Francisco in 1906. That observation has since been corroborated by many architects and engineers. We said also that this earthquake had brought home a most severe lesson to San Francisco. After the great earthquake in 1906, the city building code was revised to insure an increased wall strength through which to meet horizontal thrusts. There was no method then known to accomplish this except in terms of wind load, and so the wind-load specification was raised in 1907 from 15 lb. to 30 lb. per sq. ft. In 1925, however, it is rather curious to note that without anybody being able to tell just when, why or how, the wind load had somehow found its

level at 15 lb. again. In addition to that, the allowable limit of maximum load for structural steel had also been increased in the meantime 12½ per cent, so that San Francisco, on that day of our visit last summer, was less well prepared to meet another shock of severity such as it had once been through, than it was in 1906.

There is a good deal of other local movement on the west coast which is real. One of the first things we did when the Carnegie Institution began studying earth movements in California was to invite the U. S. Coast and Geodetic Survey to lay a network of primary triangulation over that state of such extent and precision that no surface movement would escape attention. The reason why this was done was because after the earthquake of 1906, although it showed horizontal displacements amounting to as much as 20 ft., no one knew positively from any surveys then made whether one side of the fault moved north and the other side moved south, or whether both sides moved north or both moved south in different degrees. All we knew was that there had been movement along a well-marked fault plane. The base line to which the surveys of that time were referred was itself within the zone of movement. Because of this unforeseen circumstance we were not able to tell what the movement was after the shock of 1906. Accordingly, in the formulation of the new plans four years ago provision was made for a double line of triangles from the vicinity of Reno, Nevada, which is believed to be within the zone of greatest stability opposite San Francisco, down to San Francisco Bay, thence southward along the coast to the Mexican boundary and eastward into the Colorado River Valley, which is again a region of great stability geologically.

Among other facts revealed by that survey was one of outstanding interest, in view of the disturbance at Santa Barbara last summer, namely, that the mountain station at Gaviota in the Santa Ynez Range, just northwest of the city of Santa Barbara, had moved northward about 24 ft. since the last survey of that region, a little over thirty years ago. If that means anything in terms of elastic materials it means an accumulating strain, and if earthquake theory or experience can aid at all in its interpretation such strains will eventually reach the elastic limit of the material and an earthquake will result. The purpose, then, of earthquake studies in that region is to ascertain the direction and the rate of accumulation of stresses in the rocks that compose the region, to ascertain with certainty where the strains are accumulating most rapidly and so to tell where the danger zone lies. That, together with the knowledge we have of the existing faults of the region, will indicate to us what we may expect in any particular spot. Santa Barbara has served to remind us in the most convincing manner of these structural weaknesses and of their danger.

Nevertheless there is a considerable amount of mental inertia here which must be recognized. The suggestion that it might be not only

important but actually profitable to study these stresses and zones of weakness were very welcome for several weeks after the earthquake, but now, within the year, people have drifted back to the old attitude which was perhaps best presented to me by a member of the Chamber of Commerce of Los Angeles when I was invited to address that body on this subject some four years ago, "I should like you to understand," he said, "that the only earthquakes in California are San Francisco earthquakes." Within the last few days there has come to my desk a letter from a well-known citizen of Santa Barbara which contains these words: "Remarks about the earthquakes of California and the probability of having more in the near future are doing the state of California, and particularly the communities which have passed through a serious quake, a lot of harm."

It is difficult, extremely difficult, in a community which wishes to look the other way, to provide for safety in building structures. It is not so difficult in a region which is less in danger to call attention to the same thing. The largest correspondence I have had during the last year has not related to Santa Barbara or to California at all, but to the possibility of dangerous earthquakes visiting New England or New York. There appears to be downright apprehension lest an earthquake upset the east coast or some part of it as it has the west coast recently. That question, which quite naturally occurs to many who are not geologists and it may occur to any of you, may be quite frankly answered. This coast geologically is old, and its adjustments are mostly in the past; they have occurred, and the structure, in consequence, is now reasonably stabilized; no great faults are known to be active in this region to the east of us. Last February a year ago, in the Saguenay River region, Canada, an earthquake disturbance occurred in which several buildings were more or less damaged, none of them severely. There was at Charleston, some years ago, an earthquake of very considerable devastating power, the origin of which has never been at all well known. Still earlier, in 1811, a major disturbance occurred in the Mississippi Valley to the south of us called the New Madrid earthquake, which is now being studied by Professor Macelwane, of St. Louis University.

These cases are not so clear but there has been no evidence of accumulating strain and periodic release in these regions such as has been found on the west coast. There, the study of the earth movements has plainly revealed that where an active fault line is in plain view one must build in expectation of a thrust at the level of the ground or somewhere near the foundations of the structure. If the structure is high and its center of gravity can be established, its elastic constants must be appropriate to take up the vibration which will surely strike and often from a direction which can be pretty well forecast. On the other hand, if the structure is low, it is rather easy to provide that it shall move as a whole when the horizontal shock reaches it.

In the report of the State Earthquake Commission of California in 1908 there are accurate directions how to construct a building which shall

be proof against earthquakes and I know of nothing that has happened since which would detract from the truth of that description. A heavy cement foundation with a well-braced wooden structure bolted to it, with sheathing in place of plaster wherever possible, is probably earthquake-proof. These plain directions were adapted to the climatic conditions of California and would have saved any private householder, who chose to take advantage of them, the danger from an earthquake visitation. Nevertheless, the number of such houses visible there today plainly indicates how little attention was actually paid to them. It is only possible to bring these things to your attention. It is your province, as architects and engineers, and not mine, to carry out the details of their application, and I dare say you will recognize the fact, if you have not come into personal contact with it, that if you are to build a structure on a foundation which is threatened with a horizontal thrust, that it is just as essential to take account of that thrust as it is to take account of the difference between mud and rock bottom for a foundation.

I dare say any one of you could construct a building which would meet perfectly all the conditions which obtain in any of those known localities which are particularly subject to earthquakes. Nevertheless, it is not being done, though some promising things are contemplated. There is, just off the press, a new building code for Santa Barbara. In that building code there is very complete provision both for foundations and superstructure in buildings subject to earthquakes, and that building code has had the approval of the city. Furthermore, the city government of Santa Barbara has circulated this code up and down the coast for the benefit of all who may wish to take advantage of the studies which have been made at Santa Barbara. I think, therefore, that a recurrence of this experience which is to be expected, by virtue of the fact that the region nearby has been displaced by as much as twenty-odd feet in the last 25 or 30 years, will find Santa Barbara ready to meet it. It was not ready before. Such action is not only eminently praiseworthy but likely to have its influence up and down the coast from the Mexican boundary to the Oregon line.

Seven Years of Experience With Job Control of the Quality of Concrete.

By Roderick B. Young.*

In 1917, the Hydro-Electric Power Commission of Ontario, being confronted with a large construction program, became interested in better methods for the production of concrete. About this time too, they decided to abandon the practice of stating the quality of concrete in terms of its proportions and to substitute therefor a classification based on compressive strength at 28 days.

Immediately after this was decided upon, there arose the question, "How, on any job, can the materials available be proportioned to give concretes of the different classes?" A study of the methods proposed up to that time showed none giving better than an approximate answer to this question, while laboratory tests, sufficiently extensive to furnish the desired information, seemed impracticable except for jobs of unusual size and importance. To the writer fell the task of finding a more satisfactory solution to this problem.

At about this time, L. N. Edwards was carrying out his experiments on the surface area method of proportioning[1] and the writer was fortunate in being in a position to follow this work closely. To him, Mr. Edwards seemed the first investigator to appreciate the practical side of the problem and to have a method that was simple and workable. Unfortunately, his tests had been largely confined to mortar mixtures and his conclusions could not safely be applied to concrete without further experimentation, which was undertaken by the commission. But before this could be done, Professor Abrams published the first account of his discovery of the water-cement-ratio-strength relation.[2] The practical possibilities for predetermining and controlling the strength of a concrete offered by this, were at once seen and a little study convinced the writer that the conclusions of Abrams and Edwards agreed in all essential features[3] and taken together, offered a simple and practicable solution to the problem of proportioning a concrete to a predetermined strength. Laboratory tests carried out during the winter of 1918-1919 confirmed this view.

Early Methods of Proportioning.

As a result, a method combining the ideas of both Edwards and Abrams was developed and tried out in the field the following summer. This

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method,[4] while not now used by the Hydro-Electric Power Commission of Ontario, will be described at some length, because from it sprang the present method and because it was used on the first two jobs on which scientific control was attempted. In order to understand the basic principles of this early method, it will be necessary to consider the theories of Abrams and Edwards, on which it is based.

Abrams claimed[5] that for concretes workable when molded, the compressive strength was related to the ratio of water to cement in the freshly-placed concrete mixtures, or to their water-cement ratio, each water-cement ratio having a corresponding compressive strength.

Edwards found[1] that when cement was proportioned according to the surface area of the aggregate and sufficient water was used to bring the mixtures to the same consistency, the resulting mortars had the same compressive strengths. It followed, therefore, that for mortars of equal strength and consistency, not only the quantity of cement but also the quantity of water must vary as the surface area of the aggregate.[6]

By the surface area of an aggregate was meant the sum of the areas of the surfaces of the individual particles composing that aggregate, and this value was determined from its sieve analysis by the use of values of unit area derived from studies on sand grains of different size.

To meet these conditions, recourse was had in the first case to the relation that exists between strength and water-cement ratio in plastic mixtures and in the second to the fact that the quantity of cement and water necessary to bring a mixture to a given degree of workability depends in part on the surface area of its aggregate. Accordingly concretes were proportioned to have a definite water-cement ratio and a fixed relation between cement and the surface area of its aggregate. The data necessary to do both were obtained from a simple series of tests in which mixtures of the same consistency were made up, using different ratios of cement to surface area. The proper amount of fine and coarse aggregate was determined by judgment or test, on the assumption that the best mix was one that contained the greatest amount of coarse aggregate and remained workable. This gave mixtures with approximately one-third fine aggregate, which agreed with the ideas then prevailing as to best practice.

In applying these data in the field, the total surface area of the batch was calculated, using the sieve analyses of the aggregates, their weights per cubic foot in the condition in which they were to be measured, and the ratio of fine to coarse aggregate decided upon. The total surface area thus obtained was multiplied by the cement area factor for the class of concrete being designed. This gave the field proportions and if sufficient water was added to give the proper water-cement ratio, a mixture of a standard consistency was produced. If wetter or drier mixtures were wanted, water and cement were added or left out of the batch as necessary, keeping the water-cement ratio constant. Neither cement nor water was changed separately, but both together, and in this respect the method was different from any then proposed.

Another difference was the omission of the slump test, which at the time was beginning to have considerable vogue. This was largely because experiments showed it to be impracticable with the crushed rock aggregates and lean mixtures used by the commission. It therefore became necessary to develop a method of proportioning and control that did not depend on this type of test for its success and this was done.

FIRST FIELD APPLICATION OF CONTROL OF CONCRETE.

This method of designing concrete mixtures was first tried in 1919. The commission was then building a small hydro-electric project known as the High Falls development. Measured by the amount of concrete, this was a small job—the total volume of all classes being about 6,200 cu. yd.—but because it was probably the first attempt to apply the new methods of proportioning in the field and also because of the lessons it taught, this piece of work merits a brief description.[7]

The main structures at High Falls were a gravity dam with sluiceways and a concrete powerhouse. The equipment used for concreting was of the simplest. The bulk of the concrete, that on the dam and related works, was mixed with a portable mixer and placed by a push car travelling on a trestle; the concrete for the powerhouse was handled by a stationary mixing plant on a nearby hill, from which it was distributed by chutes. The aggregate for the former was measured in wheelbarrows and for the latter in a simple charging bin. Much of the concrete was placed during the winter and the ordinary precautions of winter concreting were taken. The aggregate was a pit-run gravel containing an excess of coarse sand, mixed with crushed rock to improve its grading.

Eighty-four groups of test specimens were taken during the placing of this concrete (Table I). The average strength as obtained from these tests agreed well with the design strength of the concrete but the individual tests were quite variable. To show the character of the variations, the results of the different groups, stated as percentages of the strengths for which the concrete was designed, are plotted in sequence in Fig. 1. For about a month the tests fluctuated closely above and below the design strength, from then on until cold weather set in they exceeded the design strength; they then took a sharp drop from which they were only beginning to recover when the job ended.

The first group represents a period in setting the proportions during which no allowance was made for the moisture contained in the aggregate. The second represents the normal performance for this job, the third, cold weather operation when the cooler temperatures so affected the curing of the test specimens as to lower the results. The proportions and water-cement ratios of the last two periods were identical, and the only difference in concreting was in the heating of the materials before mixing. But the curing conditions of the test specimens were different in the two cases. When cold weather set in, it was difficult to maintain them at the proper

temperature in the temporary buildings in which they had to be stored, and it was impossible to protect them from the cold weather during their long journey to Toronto. Under these circumstances, it was felt that the specimens did not represent the concrete, it being undoubtedly of a better quality than the tests would indicate.

Four conclusions were reached as a result of the experience on this job:

1. It was possible to predetermine the proportions required to give concretes of a specified strength and workability.

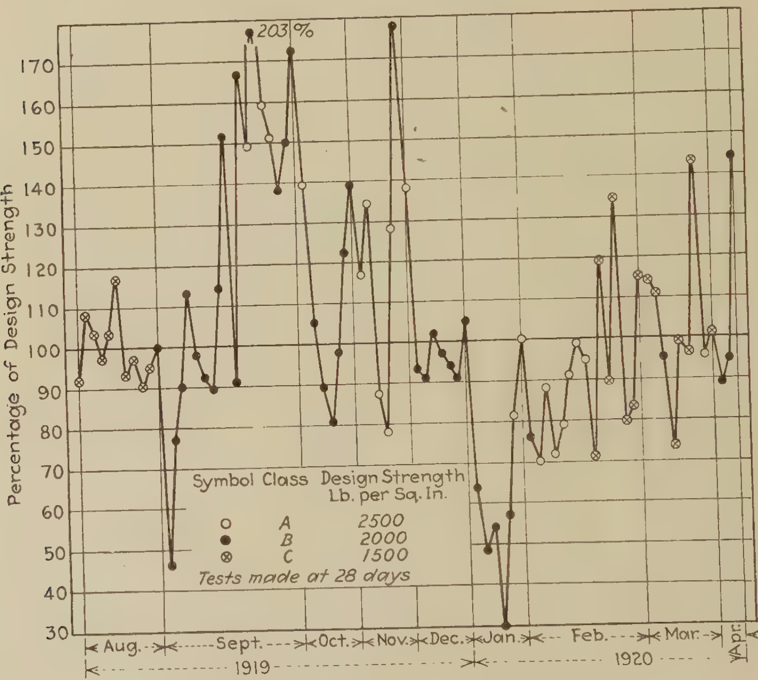

FIG. 1.—FIELD TESTS, HIGH FALLS DEVELOPMENT.

2. The combined method of surface areas and water-cement ratio used on this job was satisfactory for this purpose.

3. Test specimens should be kept at a proper temperature while curing if they were to indicate whether or not concrete of the specified quality was being obtained.

4. During cold weather, shipping test specimens long distances was impracticable.

SECOND APPLICATION—THE NIPIGON DEVELOPMENT.

The next work on which the method was used was in the construction of the Nipigon development[8] north of Lake Superior. A small laboratory

TABLE I.—SUMMARY OF 28-DAY COMPRESSION TESTS ON CONCRETE

(Specimens taken in field from 1919 to 1925, inclusive.)

Name of Job	Summary All Classes				Class A $fd=2500$			Class B $fd=2000$			Class B-C $fd=1750$			Class C $fd=1500$			Class D $fd=1000$		
	No.	$\%fd$	V	V_1	No.	fc	V	No.	fc	V	No.	fc	V	No.	fc	V	No.	fc	V
High Falls Development	84	104	22.1	60	20	2,690	23.8	39	2,060	27.2	..	....	....	25	1,510	12.7	..	....	....
Nipigon Development:																			
Power House	107	106	18.5	67	52	2,400	20.8	27	2,040	19.8	..	....	....	28	1,940	12.9	..	....	....
Main Dam	59	100	19.7	58	18	2,240	19.3	10	2,140	23.8	..	....	....	31	1,560	18.6	..	....	....
Chippawa—Queenston Development:*																			
Screen and Power House	167	115	12.0	83	26	2,530	10.0	60	2,360	11.6	18	2,000	11.0	63	1,790	13.5	..	....	....
Canal and Forebay	318	117	19.5	66	..	....	....	13	2,150	14.9	..	....	....	276	1,740	19.3	29	1,140	23.0
Intake	19	116	6.0	100	8	2,760	6.5	3	2,040	1.2	..	....	....	2	1,520	5.6	6	1,340	7.9
Ranney Falls Development	106	81	21.7	54	13	1,590	11.9	64	1,710	22.3	..	....	....	29	1,180	24.9	..	....	....
Dam 8 Development	32	96	19.3	62	9	2,380	10.9	7	1,920	15.3	..	....	....	16	1,440	25.7	..	....	....
Dam 9 Development	31	95	13.3	81	18	2,230	9.0	6	1,990	9.6	..	....	....	7	1,580	27.8	..	....	....
Extension to Nipigon Power House:																			
Units 3 and 4	29†	10.4	15.3	76	20	2,510	16.8	2	2,400	16.7	..	....	....	7	1,770	10.7	..	....	....
Units 5 and 6	82	98	15.5	75	36	2,430	13.2	24	2,020	18.3	..	....	....	22	1,460	16.4	..	....	....
Extension to Queenston Power House:																			
Units 6 to 9	86	108	6.2	97	12	2,710	5.6	56	2,150	6.2	11	1,830	5.9	7	1,780	8.3	..	....	....
Hanna's Chute Development	15	142	15.2	67	6	3,510	13.9	5	2,670	13.5	..	....	....	4	2,310	19.5	..	....	....
Virgin Falls Storage Dam	34	115	15.1	85	5	3,800	19.8	23	2,130	12.5	..	....	....	6	1,770	21.1	..	....	....
Repairs to Tunnel, Toronto Power Co.	3	112	2.0	67	2	2,660	1.5	..	....	....	..	....	....	..	....	....	1	1,220	....
Sunnyside Tower Footings	4	119	4.5	100	..	....	....	..	....	....	..	....	....	4	1,780	4.5	..	....	....

* Eleven bridges built. Results cannot properly be summarized.

† Forty groups of tests made. Owing to failure of testing machine, eleven sets could not be tested at 28 days.

No. = Number of test groups made.

fd = Design strength, lb. per sq. in.

fc = Average compressive strength, lb. per sq. in.

V = Mean variation in per cent.

$\%fd = \frac{fc}{fd} \times 100$ (weighted).

V_1 = Per cent of number between 80 and 120 per cent fc.

was installed here, equipped for crushing the field specimens, and an experienced concrete inspector was assigned to the staff of the resident engineer to run this laboratory and supervise the concreting. In this way it was possible to control the field testing and to have the inspection cover every detail from the materials to the finished concrete.

The original construction required the placing of approximately 38,000 cu. yd. of concrete. This was handled by two stationary plants equipped

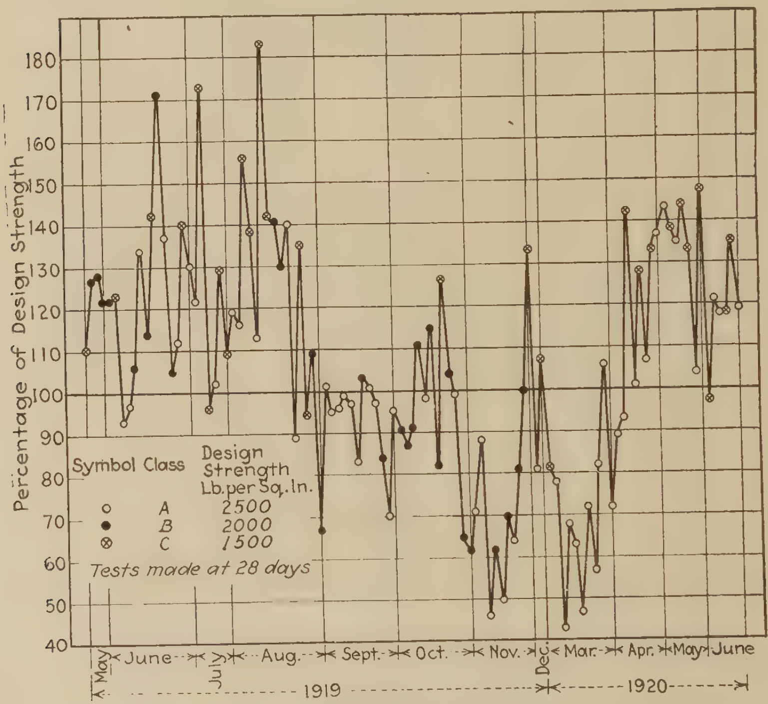

FIG. 2.—FIELD TESTS, NIPIGON DEVELOPMENT.

with overhead bins, measuring hoppers and twin mixers. The concrete was distributed almost entirely by chutes.

Conditions were not ideal for applying any method of proportioning. The cement situation in the summer of 1920 was chaotic and it was necessary to use six different cements varying considerably in their characteristics. Sand was obtained from two different sources, a gravel pit and a sand pit. That from the sand pit was fine and micaceous, that from the gravel pit so dirty that it required washing. When available, the latter was mixed with the pit sand to improve its grading. The variations in the materials were great, were difficult to anticipate and compensate for, and

added materially to the problem of proportioning the mixtures accurately. To further complicate matters, labor was inefficient and hard to manage and the job was being rushed.

Concreting began in May 1920 and continued until June 1921. One hundred and seven sets of test cylinders were taken, the results of which are summarized in Table I and plotted in Fig. 2. As at High Falls, a falling off in strength occurred during the cold weather. This was thought at first to be due to some other reason than low curing temperatures but when the strengths rose again in the spring, there seemed no other reasonable explanation. While every effort was made to cure the cylinders at the proper temperature, it was found to be practically impossible to do so under the conditions and with the facilities available.

It was on this job that the practice of crushing one of each group of test cylinders when 7 days old was originally tried. It was found to give useful information and the practice has since become standard.

The Nipigon development afforded the first opportunity to study the other factors entering into the quality control of concrete, that is the control and handling of materials, mixing-plant operation, placing and curing, and as a result, the conclusion was reached that the design of the proportions was only one step and possibly a minor step, in the production of a quality concrete. Another important conclusion was that variations in the concrete were largely due to variations in the quantity of aggregates, particularly the fine aggregate, brought about by the methods used in measuring.

THE CHIPPAWA-QUEENSTON DEVELOPMENT.

During the same summer, preparations were made to apply similar methods on the Chippawa-Queenston development[1] then under construction near Niagara Falls. A field laboratory was again installed and as at Nipigon, inspection was complete and included every step from the sources of the materials to the finished concrete. Concreting operations were not commenced on a large scale until November, 1920, but during the previous summer, scientific control was applied to minor operations in order to familiarize the organization on this job with the details of the methods to be used.

An important outcome of this preliminary work was the decision to change the manner of stating water-cement ratio. Up to this time it had been the practice, as it is still the practice with most engineers, to define water-cement ratio as a ratio of volumes. But A. C. D. Blanchard, then chief field engineer of the Chippawa-Queenston development, suggested that water-cement ratio could be more conveniently stated as a ratio of weights and this proposal had so much merit and appealed so strongly to those who were actually doing the proportioning that it was adopted. There has been no reason to regret this step.

Excepting the Wilson Dam, the Chippawa-Queenston development is, we believe, the largest job on which control of concrete has ever been ap-

plied, and in the size and diversity of the concreting operations, it is probably unique amongst big concreting operations. Briefly, the work consisted of an intake structure in the Niagara River at Chippawa, above Niagara Falls on the Canadian side, a screenhouse and powerhouse at the edge of, and in the Niagara River gorge, and a 12¾-mile canal connecting the two. The screenhouse, powerhouse and intake were all of concrete and the canal was concrete lined for 7½ miles of its length. Besides these major structures, there were a number of railway and highway bridges, all requiring concrete. The total amount involved was about 500,000 cu. yd.

TABLE III.—CONCRETE PLACED ON CHIPPAWA-QUEENSTON POWER DEVELOPMENT.

(Week commencing June 20, 1921)

Plant	Cubic Yards Placed
Powerhouse	1,162
Screenhouse	1,319
Forebay	151
Forebay (Gunite)	40
Division III:	
No. 1 Lining Plant	998
No. 2 Lining Plant	137
No. 3 Lining Plant	1,131
No. 4 Lining Plant	1,306
No. 5 Lining Plant	1,866
No. 6 Lining Plant	2,170
Curve Lining Plant	336
No. 1 Paving Plant	137
No. 2 Paving Plant	219
No. 4 Paving Plant	172
No. 5 Paving Plant	966
Montrose Stationary Plant A	1,381
Whirlpool Stationary Plant	4,787
Total	18,278

Concreting on the screen and powerhouses commenced in 1920 and continued into 1922 when the installation of the first five units was completed. Concreting of the canal lining was begun in November 1920 and completed in December 1921. The former was handled by stationary plants, the latter by travelling as well as stationary plants, ranging in size from a central mixing plant with a capacity of 1,000 cu. yd. per day to small portable plants placing concrete at special points. Concrete was handled by chutes, buggies and by train; in fact the job offered such a variety of conditions under which concrete had to be handled that the method and plant used varied greatly for different parts of the work.

The aggregates used were sand and crushed stone. The sand came from several sources, from pits nearby and from Lake Ontario. The stone

was a dolomitic limestone taken from the canal excavation and was crushed on the job. Cement came from five different sources and, as at Nipigon, varied in its properties. To add to these difficulties, the work was being rushed day and night, and with the exception of the chief field inspector, the engineering staff had had little experience with the methods used.

Some idea of the magnitude of the concreting operations on this job may be gathered from the following: From Nov. 1, 1920, to Dec. 17, 1921, 410,000 cu. yd. of concrete were placed, 379,000 cu. yd. of this being in 1921. Of this quantity, 332,000 cu. yd. were deposited from May to December inc., or at an average rate of 41,500 cu. yd. per month. The record day's pour was 3,046 cu. yd. with 17 plants operating, the record week, 18,278 cu. yd. and the record month, 63,362 cu. yd. An idea of the diver-

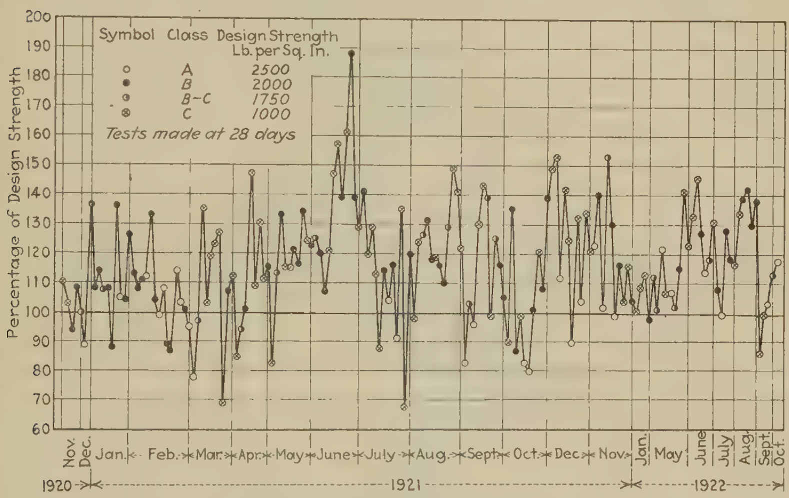

FIG. 3.—FIELD TESTS, POWER HOUSE, SCREEN HOUSE AND RELATED WORKS, CHIPPAWA-QUEENSTON DEVELOPMENT.

sity of operations is obtained from Table III, which is the record of concrete placed during the week of greatest production.

Referring again to Table I, it is seen that over 500 groups of test specimens were taken during the concreting of the Chippawa-Queenston development, and for convenience they have been arranged according to the major divisions of the work such as screen and power houses, canal and forebay, etc. While the intake, the later bridges and the extension to the screen and power house are part of the completed development, they were built after the plant went into operation and hence are more properly considered separately.

In Figs. 3 and 4 are plotted the bulk of these 500 tests, Fig. 3 for the screen and power houses, Fig. 4 for the rock section of the canal. In

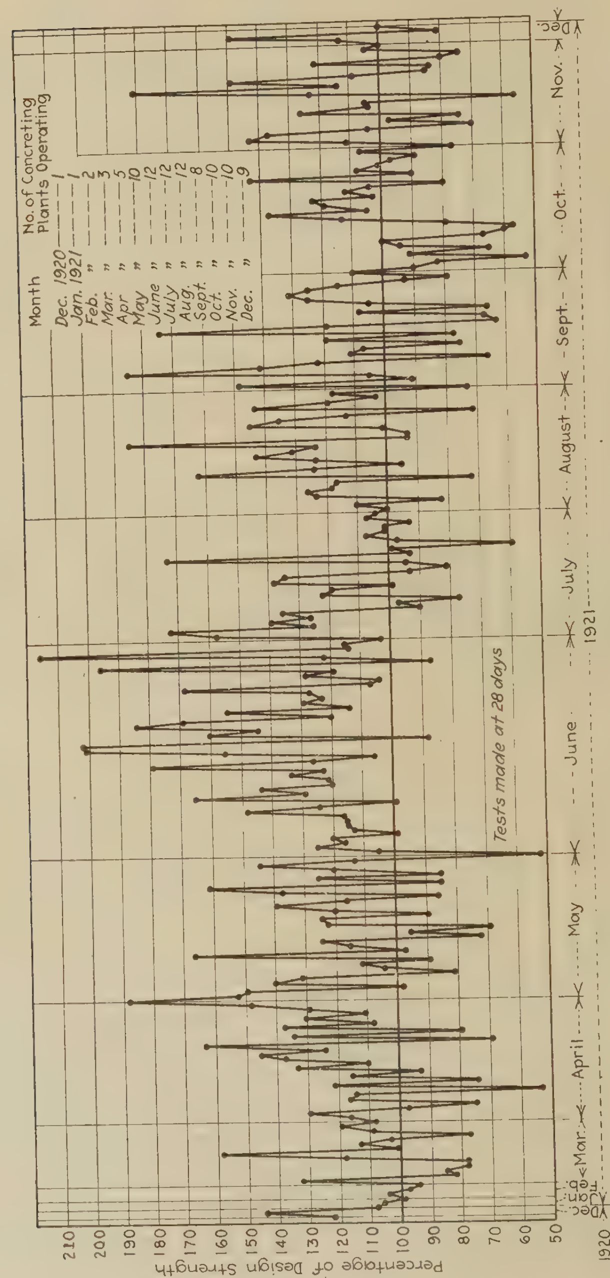

FIG. 4.—FIELD TESTS, CANAL LINING IN ROCK, CHIPPAWA-QUEENSTON DEVELOPMENT.

neither case has any attempt been made to segregate results by concreting plants, the tests being plotted in sequence irrespective of their origin. In studying them, therefore, it should be remembered that Fig. 3 includes results from three plants, closely situated, while Fig. 4 includes results from 19, of which 14 were travelling plants operating along the bottom of the canal, and that the tests were made by many different inspectors, few of whom had previous experience in taking and sampling concrete. Neither of these curves shows any appreciable dropping off in strength due to winter concreting, while in the case of the stationary plants of the screen and power houses, where conditions were favorable, the mean variation is much less than any previously obtained.

SURFACE AREA CALCULATIONS ABANDONED.

The summer of 1921 saw the methods of determining proportions in the field considerably modified. The surface area calculations were dropped although they still formed part of the preliminary laboratory studies. This change came about gradually. At Nipigon, in 1920, it was found that the mixtures could be entirely controlled by regulating the water content and that only occasionally did there arise conditions requiring readjustment of the basic proportions. The experience in 1921 confirmed this and showed further that when changes in the mixture did become necessary they could be made in the field without recalculating proportions, by restoring the mixtures to their original consistency, adding or omitting cement and water as required to maintain the water-cement ratio for the class of concrete in hand.

RECENT APPLICATIONS OF JOB CONTROL.

During 1922 and 1923, control methods were applied to a number of jobs, including 7 bridges along the Chippawa-Queenston Canal. There was nothing about this work to warrant special mention. Probably the most important points bearing on the control of concrete brought out by these jobs were the desirability of having uniform materials and the importance of placing.

During this same period, considerable thought and study were given to the design and economy of mixtures and certain experimental work conducted. As a result, methods of proportioning were modified and greater attention was given to the measurement of aggregate and to securing uniform materials. Also the commission decided to experiment with the weighing of aggregate and in 1924 installed a plant of this type. The experiment has been a marked success both from the standpoint of control and economy.

During 1924 and 1925 the methods at present in use by the commission were applied on six jobs, including the extension to the power house for Units 5 and 6 at Nipigon, on which the weighing plant referred to above was used. The test results for two of these jobs have been plotted as before in Figs. 5 and 6. The results here, as in the previous cases, have been

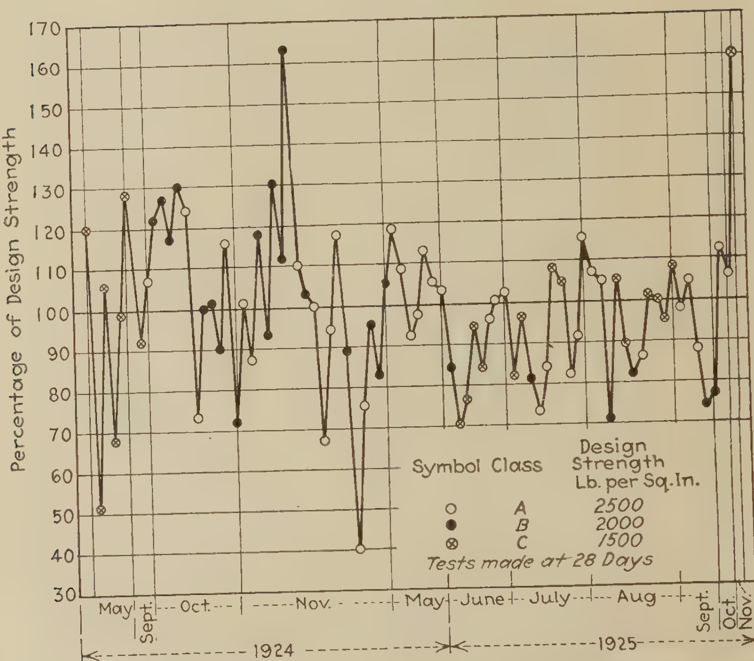

FIG. 5.—FIELD TESTS, EXTENSION TO NIPIGON POWER HOUSE, UNITS 5 AND 6.

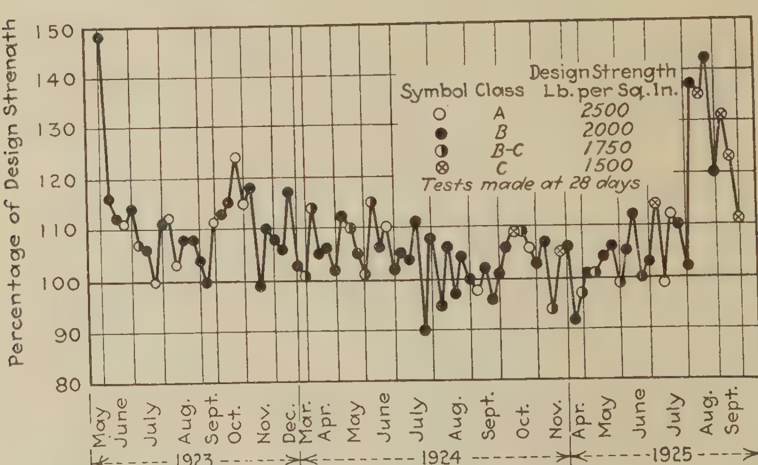

FIG. 6.—FIELD TESTS, EXTENSION TO POWER AND SCREEN HOUSE, UNITS 6 TO 9, CHIPPAWA-QUEENSTON DEVELOPMENT.

influenced considerably by local factors. At Nipigon, trouble developed with the cement. Two brands were used, one of which gave some 5 per cent less strength than the other. The result of this was noticed at once, but the cause was not definitely located for some time so that a remedy could be applied. The effect on the test results was to lower the average strength and to increase the mean variation.

On the second of these, the extension at Queenston, the mixing plant was the same as used for the completion of the earlier work. The aggregates were sand and crushed rock of excellent quality and were measured by volume. The concrete was handled from the mixer to the work by chutes, several hundred feet long, and distributed, where necessary, by

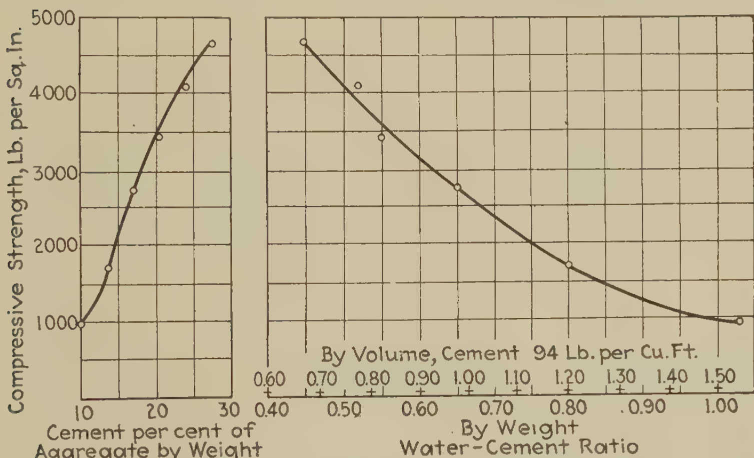

FIG. 7.—DATA OF TABLE III, PLOTTED TO ILLUSTRATE CHARACTER OF TESTS MADE BY THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO IN OBTAINING INFORMATION FOR DETERMINING FIELD PROPORTIONS.

buggies. Conditions on this job were favorable to uniform concrete as is evident from the tests obtained.

Altogether the Hydro-Electric Power Commission of Ontario has attempted to control the quality of concrete on 22 jobs, one of which, the Chippawa-Queenston development was itself made up of a number of distinct operations. Table II shows the magnitude of these and the variety of conditions which have been encountered. These jobs have included powerhouses, dams, retaining walls, concrete arch bridges, bridge piers, foundations and other works. They have included mass and reinforced concrete and the reinforced concrete has been of both heavy and light construction. They have included work done by the forces of the commission and work done by contract. And on these jobs there has been encountered

TABLE II.—CONSTRUCTION OPERATIONS ON WHICH CONTROL METHODS WERE APPLIED TO CONCRETE.

Name of Work	Situation	Year Built	Duration of Concreting	Concrete in Structures, cu. yd.	Built by	Character of Aggregate	Type of Concreting Plant: Mixing	Placing
High Falls development	Mississippi River	1919–20	10 mo.	6,210	Hydro	Pit run gravel with crushed rock added	Portable and stationary	Cars, chute and buggies
Nipigon development:								
Powerhouse........	Cameron Falls, Nipigon River	1920–21	14 mo.	38,060	Hydro	Sand and gravel or crushed rock	Stationary	Chutes
Dam..............	Cameron Falls, Nipigon River	1921	4 mo.	11,400	Hydro	Washed pit run gravel	Stationary	Chutes
Chippawa—Queenston development:								
Screen and power-house, units 1–5..	Niagara River and near Niagara Falls, Ontario.	1920–22	24 mo.	114,580	Hydro	Sand and crushed rock	Stationary	Chutes and buggies
Canal and forebay..	Niagara River and near Niagara Falls, Ontario.	1920–21	13 mo.	310,740	Hydro	Sand and crushed rock	*	Chutes, trains and buggies
Four bridges.......	Niagara River and near Niagara Falls, Ontario.	1920–21	†	18,060	Hydro	Sand and crushed rock	Portable and stationary	Various
Seven bridges......	Niagara River and near Niagara Falls, Ontario.	1922–23	†	11,280	Contract	Sand and crushed rock	Portable	Various
Intake............	Niagara River and near Niagara Falls, Ontario.	1922	5 mo.	33,010	Contract	Mixed river gravel	Stationary	Trains and chutes
Ranney Falls development.........	Trent River	1921–22	10 mo.	9,250	Hydro	Washed pit run gravel	Stationary	Chutes
Dam 8 development ...	Trent River	1923–24	8 mo.	3,860	Contract	Pit run gravel	Stationary	Cars and chutes
Dam 9 development....	Trent River	1924	6 mo.	3,460	Hydro	Pit run gravel	Stationary	Chutes
Extension to Nipigon powerhouse:								
Units 3 and 4......	Cameron Falls, Nipigon River	1923–24	9 mo.	9,810	Hydro	Pit run gravel	Stationary	Chutes
Units 5 and 6......	Cameron Falls, Nipigon River	1924–25	11 mo.	10,720	Hydro	Sand and washed gravel	Stationary‡	Chutes
Extension to Queenston powerhouse:								
Units 6–9..........	Niagara River	1923–25	27 mo.	52,010	Hydro	Sand and crushed rock	Stationary	Chutes
Hanna's chute development	Muskoka River	1925	6 mo.	2,430	Hydro	Pit run gravel	Semi-stationary	Cars and chutes
Virgin Falls storage dam	Nipigon River	1925	7 mo.	2,830	Hydro	Sand and crushed rock	Portable	Buggies and chutes
Repairs to tunnel, Toronto Power Company	Niagara Falls	1925	3 wk.	2,230	Hydro	Sand and crushed rock	Stationary	6-in. pipe and trains
Sunnyside tower footings	Toronto, Ont.	1925	9 wk.	860	Hydro	Sand and crushed rock	Portable	Wheel-barrows
Total.............	....	....	....	640,800				

* Includes 20 different concreting plants and included portable, stationary and traveling.
† Individual jobs of different durations.
‡ Aggregates measured by weighing.

a variety of materials and equipment, and the personnel has included men sympathetic, men indifferent and men openly antagonistic to the methods and ideas on concreting held by the engineers of the commission.

Table I summarizes the test results obtained on these jobs excepting only those for the bridges across the Chippawa-Queenston Canal, the results of which did not lend themselves to a summary. A study of this table shows that, in general, the average compressive strengths obtained were close to the design strengths. The exceptions are due to special circumstances. For instance, at Ranney Falls, the principal cause was low temperature curing of the test specimens for no testing machine was available for this job and the field specimens had to be shipped to Toronto. As most of the concreting was done in the winter, low tests were to be expected. A contributing factor was an unsatisfactory aggregate, a pit-run gravel deficient in very fine sand, that segregated badly in handling and was hard to control. A similar aggregate affected the results on the developments at Dams 8 and 9, built in the same locality, although on these, constructed later, steps were taken to counteract its objectionable features.

In the other case, the development at Hannas Chute, the pit-run gravel used proved more uniform and required less cement than expected. To have reduced the cement a half bag per batch, the smallest reduction which is ordinarily practicable, would with the small capacity concreting plant used, have required an undesirable reduction in the size of the batch, so that the mixtures were purposely over-designed. In spite of this a substantial saving in cement was effected.

The mean variation obtained for these jobs ranges all the way from 2 per cent to 22.1 per cent, but the relative uniformity of the concrete of the different jobs is not reflected in the magnitude of these mean variations. For example, the tests for the intake structure of the Chippawa-Queenston development show a mean variation of but 6 per cent, less than for the extension for Units 5 and 6 at Nipigon, yet the concrete on the former is known to have been more variable in quality than on the latter. On the three jobs, Ranney Falls, Dam 8 and Dam 9, the mean variations are in the order of the relative uniformity but the concrete at Dam 9 with a mean variation of 13.3 per cent was probably not as uniform as that at Virgin Falls with a variation of 15.1 per cent. The concrete on the extension to the Queenston powerhouse was better than on the original construction but on neither of these jobs was the concrete more uniform than at Virgin Falls or on the extension for Units 5 and 6 at Nipigon. Thus a knowledge of the job and more particularly of the facilities for curing and testing the specimens is necessary in the interpretation of field results.

We have referred in the first part of this paper to laboratory studies made prior to the application of scientific proportioning at High Falls. Such experimental studies have been an important feature in the development of the methods used by the commission. Each question as it arose in the course of the field work, has been investigated, and while it is impossible to go into this work in detail, some reference to it is necessary in

order to give a proper understanding of the reasons for adopting the methods to be described later.

Investigation of Proportioning Theories.—One of the earliest questions to be studied arose out of the controversy between Abrams, Williams and Davis regarding the water-cement-ratio-strength relation.

At the same time, a study was made of both the surface area and fineness modulus methods of computing proportions. Reasoning further, it appeared that if by using surface area or fineness modulus, mixtures of a uniform consistency resulted, the cement and water would therefore, of necessity, vary in accordance with these factors no matter what the change in materials, if the mixtures were arbitrarily kept at a uniform consistency and water-cement ratio. It was thus possible to comply with the theoretical requirements of these methods without actually using them and the conclusion was reached that they were not therefore of particular importance. In any case the water-cement-ratio-strength relation governed and was alone sufficient for proportioning mixtures.

Another conclusion was reached, namely, that the proper function of these theories was for determining the best and most economical combination of cement and aggregate for a given set of conditions, but since other factors besides strength needed to be considered in designing concrete, arbitrary methods of proportioning such as these, should be used with caution and were, even for this purpose, of limited application.

THE PROPER QUANTITY OF FINE AGGREGATE IN CONCRETE MIXTURES.

The subject of economical mixtures brings up another point upon which considerable investigation has been and is still being carried on. The most economical concrete has long been considered one containing as much coarse aggregate as the mortar would carry, and the general practice has been to have one volume of fine aggregate for each two volumes of coarse. With aggregates measured moist, as in the field, this ratio gives mixtures in which the fine aggregate is less than 30 per cent by weight of the total inert material. In our early studies with the materials with which we were then dealing, this amount of fine aggregate was about as little as could be used and still have the requisite plasticity and for the first few jobs, therefore, the aggregates were arbitrarily proportioned on this basis.

In our early experiments it was found that with average well-graded materials, the aggregates had to contain about 40 per cent by weight of sand in order to give test pieces free from pitting. It was also found that the less sandy mixtures, which were harsh working in the small quantities experimented with in the laboratory were quite workable in the large masses handled in the field. But in the field when workability was a matter of special importance, it proved advantageous to increase the amount of fine aggregate over that ordinarily used and it was noticed that, contrary to expectations, this did not always increase the cement used per

cubic yard of concrete. The point thus raised was so important that an explanation was sought and there has gradually developed an investigation on the economics of concrete mixtures, which is still uncompleted.

For the purposes of this paper we need to refer only to two points brought out by this investigation. The first is that for each combination of fine and coarse aggregate, there is a ratio for which the cement content of the concrete is a minimum and this mixture contains more fine aggregate than is required for plasticity alone. The second is that the sand content, except for very fine sands, can be increased several per cent above the minimum point without appreciable change in the cement content of the finished concrete.

MEASUREMENT OF AGGREGATES.

The measurement of aggregates has also been investigated. The purpose of measurement as applied to aggregates is to obtain the same volume of solids or absolute volume of material in each batch. With granular materials, the same measured volumes do not necessarily give the same absolute volumes, for the solids in a measured volume depend upon the method of filling the container, the grading of the aggregate, and in the case of fine aggregates, on the moisture content. Thus any volumetric method of measuring aggregate, no matter how carefully made, is liable to errors from these three sources.

Our studies showed first, that volume changes of from 20 to 40 per cent could result when moisture was added to fine aggregate but in the field the error from this source would ordinarily be less than 5 per cent. Secondly it was probable that variations in grading caused errors equally as great as those due to changes in moisture and, thirdly, there would always be unavoidable variations in filling the container, even in a proportioning plant operating under the best of conditions.

Measurement by weight seems to be the only method whereby these errors can be eliminated, for since the specific gravity of an aggregate is practically a constant, its weight varies directly as the volume of its solids and the errors of measurement are those due to balancing the scales and to moisture in the aggregate. With a properly operated weighing plant, the former are very small, a fraction of 1 per cent in the experience of the writer, while the latter, unless absolutely ignored, causes errors of less than 1 per cent.

PRESENT METHODS OF THE HYDRO-ELECTRIC POWER COMMISSION.

The system of control now followed by the commission starts with the raw materials from which the concrete is made. Cement is inspected at the mills prior to shipment and no cement is used that has not met all requirements of the specifications up to and including the 7-day mortar test. The aggregates are checked continuously for quality and grading, at the source of supply if possible, if not, when received, and the water is tested when it comes from a source of doubtful quality.

The second step in the system of control is the setting of the mixtures. The method used at the present differs from that first adopted; it depends upon the water-cement-ratio-strength relation alone and functions through control of the water. In practice it is carried out as follows:

Before concreting commences, proportioning tests are made in the laboratory with the cement and aggregates to be used on the job in question. This is done on all but the smallest jobs, those of less than a thousand cubic yards of concrete. This series of tests consists of 5 or more groups of five 6 x 12-in. cylinders proportioned to give at 28 days compressive

TABLE IV.—DATA OBTAINED FROM PROPORTIONING TESTS—JOB A.

Serial No.	Cement, per cent Aggregate	Water, per cent Aggregate	Flow	Concrete, weight per cu. ft.	Compressive Strength, lb. per sq. in.	W/C Ratio by weight	Cement, lb. per cu. ft. of Concrete
2032	25	11.3	165	151	4,650	0.45	0.277
2033	20	10.4	170	152	4,030	0.52	0.233
2034	17	9.4	154	152	3,460	0.55	0.205
2035	14	9.1	151	151	2,790	0.65	0.172
2036	11	8.8	155	149	1,690	0.80	0.137
2073	8	8.2	148	148	960	1.03	0.100

Ratio of fine aggregate to coarse aggregate: 40/60 by weight.
Fine aggregate: graded 0 to No. 4, S. A. 20, F. M. 2.60.
Coarse aggregate: graded No. 4 to 1½ in. round opening, S. A. 0.7, F. M. 7.14.

PROPORTIONING DATA—JOB A.

STATED IN CANADIAN UNITS: by Weight, Imperial Gallons.

Class of Concrete	Desired Compressive Strength at 28 days	Cement, per cent Total Aggregate	W/C Ratio	Water, gal. per bag of Cement	Bags Cement (87½ lb. gross) per cu. yd. of Concrete	
					In Test	Use
A	2,500	13.0	0.68	5.8	5.2	6
B	2,000	11.5	0.75	6.4	4.6	5¼
C	1,500	10.5	0.85	7.3	3.8	4¼
STATED IN UNITED STATES UNITS by Volume, United States Units.						
A	2,500	13.0	1.02	7.6	4,85	5.6
B	2,000	11.5	1.12	8.4	4.3	4.9
C	1,500	10.5	1.27	9.6	3.5	4.0

Use for ratio of fine aggregate to coarse aggregate: 40/60 by weight.

strengths covering the classes of concrete used by the commission. These test specimens are made in accordance with the standard requirements of the American Society for Testing Materials, excepting the proper consistency is left to the judgment of an experienced operator, no reliance being placed upon slump or flow tests, although the latter are usually made for purposes of record.

The proper ratio of fine to coarse aggregate for these tests is decided beforehand. The best ratio is largely a matter of judgment and is based upon a study of the grading of the aggregates and a knowledge of the mixtures that have given economical and workably mixtures, with similar materials.

These tests establish the water-cement-ratio-strength relationship and the cement content for the materials, consistency and proportions on the

job in question and from these are set, for field use, the limiting amounts of water and the probable cement content for concrete of the different classes. Such a group of tests and the field data derived from it, are given in Table IV.

From these data the field engineer determines the final proportions. He first makes tests to find the moisture present in the aggregates and, if the materials are to be measured by volume, their weight per cubic foot under the operating conditions of the plant. Then from the size of mixer on the job and the class of concrete required, he calculates in turn:

a. Quantity of cement per batch.

b. Total quantity of water per batch.

c. Quantity of fine and coarse aggregate per batch.

d. Quantity of water contained in fine and coarse aggregates per batch.

e. Quantity of water to be added per batch.

This done, one or more full sized batches are made up and observed. If the consistency appears too dry, cement and water are added, keeping the total water within the limits for the class in hand; if the batch appears too wet, cement and water are left out in the same way. If the mixture seems harsh, the fine aggregate is increased, if sandy, a quantity is left out. These changes may require a readjustment of the cement and water to restore the consistency. As it is ordinarily impracticable to measure cement in smaller units than a half bag, these changes may also require a readjustment in the size of the batch. In other words, the field proportions are finally determined by trial and error, to meet the two requirements of a limited quantity of water and a suitable workability.

From time to time these proportions may have to be altered to suit changes in the aggregates, but experience shows the need for this is not nearly so great as would be expected. Generally speaking, there is no necessity, either from the standpoint of control or economy, for redetermining the proportions for each new lot of aggregate. Any differences in their characteristics of sufficient consequence to warrant altering the proportions will manifest themselves in the consistency, which may be restored by changes in the quantity of cement and water, or less frequently, in the quantities or relationships of the aggregates.

Once or twice a day the engineer or inspector determines the moisture content of the aggregates and makes such changes in the quantity of gauging water as are shown necessary thereby. Ordinarily the moisture content of an aggregate does not differ appreciably from batch to batch although under certain conditions large variations are possible. Irregularity in moisture in the materials can usually be prevented or corrected for and should not be viewed as a necessary evil. When changes of any magnitude occur, they are immediately noticeable in the consistency of the concrete and can be compensated for by changing the water in the batch until the consistency is restored to the proper point.

From this on, control takes the form of close supervision over the processes of concreting, that is the measurement of the materials, mixing,

placing and curing. Of these, our experience would lead us to place most emphasis on measurement of the water and aggregates and on placing. Means of accurate measurement must be provided in the concreting plant if uniform batches are to be obtained, for a high standard of workmanship alone will not be sufficient. In placing, workmanship is more important than plant.

Few contractors or engineers see the structures they build after they are completed. If they did, there would be a more general appreciation of the importance of placing. A concrete structure may seem above criticism at the time it is finished but in a few years every patch, every corner where the mortar collected will become visible to offer mute testimony to neglect in placing. And while concrete may be made to handle more easily and be more uniform and homogeneous by a proper choice of proportions, accurate measurement and adequate mixing, in the last analysis successful placing is a matter of workmanship and supervision, not of equipment or previous treatment.

FIELD TESTING.

There is one step remaining in the control of concrete as practiced by the commission which, though referred to, has not been considered as yet. This is the taking and testing of samples of job concrete as a record of performance and as a check upon the work.

The commission places a field laboratory on all but the smallest of jobs. These field laboratories are equipped with a simple and inexpensive 100-ton hydraulic press, sieves for both fine and coarse aggregate, scales, pans, measures, molds, heaters, etc. The total cost of this equipment in Canada, where most of it has to be imported, is less than $1,000. Interest and depreciation per job run about $200, a charge that is largely offset by the saving in express otherwise incurred in shipping test specimens to the nearest laboratory.

Groups of test cylinders are taken periodically throughout concreting operations, the frequency depending on circumstances. These tests are taken more often at the start of a job than after it is well organized and more often where difficulties are being experienced than when the work is running smoothly. On the Chippawa-Queenston development it was usual to take two groups of specimens a week from each plant, although a thousand cubic yards or more of concrete might be produced in this interval, while on small work, where concreting is not continuous, test groups are often required for every pour exceeding 50 cu. yd.

In deciding upon the number of tests, it should always be remembered that testing is but a means to an end and that end is the dual one of a check upon the quality of the concrete and for purposes of record. Testing by itself cannot control the quality of concrete. That requires the regulation of the water content of the mix and the supervision of processes. The concrete is the thing and if the methods of control are giving a uniform concrete and if they and the aggregates do not change, a large number of

test pieces adds but little to the information that is to be gained from a few. On the other hand, if uniform concrete is not being produced, the test specimens which under these conditions can represent only a relatively small mass of concrete, would have to be taken in impracticable numbers to have anything like a complete record of the work. Naturally under the latter conditions, more tests should be made than under the former, but even so, it is of little advantage to take more than enough to establish the average quality of the concrete and its lack of uniformity. In fact it is always well to remember that the field inspector, who in most cases must make these tests, is better employed in inspecting the concrete and concrete materials, and that he cannot fulfil his proper function while making a multitude of tests. This is particularly true when conditions are unfavorable.

A group of test specimens consists of three cylinders, one of which is tested when 7 days old, the other two at 28 days. Whenever possible, the concrete from which the specimens are made is taken from the forms; when this cannot be done it is taken from the discharge of the mixer. Standard methods are followed in molding, curing and testing, particular care being taken in the matter of curing, for unless proper curing conditions exist the results are not indicative of the quality of the concrete.

The curing of test specimens is a difficult problem in the field. It is easy to provide damp storage but it is not so easy to maintain the required temperatures, particularly in cold weather. Even in heated buildings of permanent construction, the temperature will drop at night and in the usual temporary building in which a field laboratory must be housed, it is practically impossible always to have the proper temperature during cold weather. Under such conditions, the best that can be done is to minimize the effects of adverse curing as much as possible and to allow for it in interpreting the results.

INTERPRETATION OF FIELD TESTS.

The interpretation of field tests is a matter of some difficulty, unless the whole facts of the job are known. If the tests meet specifications, all well and good, but if they are below, it is still possible that the concrete is satisfactory. In such cases, the writer places more reliance on a knowledge of surrounding conditions,—the time of year, the method of handling specimens, the type of plant and methods of concreting,—than on the actual test results. But while indications may point to the tests being at fault, it is never safe to assume that the concrete is right without an investigation into the facts, for the low test may be and often is, a warning of some undesirable condition that should be corrected.

It is here that the 7-day tests are valuable, for while they are not as reliable an indication of quality as later tests, they yield information quickly enough to be of immediate use.

A series of tests reflects not only the uniformity of the concrete but the methods of sampling, molding, curing and crushing the specimens. For

example, specimens cured at a uniform temperature such as would exist in an established laboratory, should be less variable than those cured in the field, and specimens made and broken by the average field inspector as a part of his many duties cannot be expected to be as uniform as the work done by laboratory experts.

The mean variation of the tests will be affected by the number of specimens taken. A job on which but a few tests are made is liable to show a high mean variation if one or two of these are abnormally low or high, whereas if more tests were made, the effect of these exceptional results would be relatively less. The duration of the job is an even more important factor. In a job covering a year, the average strength of the concrete for the first six months may be lower or higher than for the second, although both may meet the specifications. This may be due to differences in cement or aggregates, to different curing conditions or to some other factor. The variations for either of these periods would be less than for the two periods combined and if the period of time was reduced still more, to say a few weeks, the mean variation should be lower still.

What should the variation be for a well-conducted job? This is a difficult matter on which to express an opinion. The writer believes that on work extending over a considerable period, say six months or more, and so situated that field laboratories must be used, concrete of more than average uniformity will have a mean variation of from 12 to 15 per cent, and that under more favorable conditions such as where established laboratories are available, this variation should be below 10 per cent for equal concrete. On jobs covering periods of but a few weeks, the average variation should be still less, say from 6 to 8 per cent.

Table V is offered in support of this opinion and as a comparison between the results obtained by the commission and by others who have attempted to control the quality of concrete on the job. In considering this table it should be remembered that, with the exception of the California bridges, every job was handy to a regular laboratory, and on the first seven jobs, the tests were made as an experiment to determine whether or not field testing and control were practicable. The testing on the latter was done by laboratory experts assigned to the jobs for this purpose only and on several, the tests covered but a part of the work on which they were taken. Under even these favorable conditions, the resulting mean variations are as high as 16 per cent.

Both Tables I and V show for each job the percentage of tests within 20 per cent of the average. This method of judging the uniformity of the concrete is subject to the same limitations as where the values of mean variation are used. In fact, as already intimated, there seems to be no factor which can be obtained from field tests which is more than an approximate indication of the uniformity of the concrete.

The average compressive strength of a series of tests is not a measure of the control obtained although it is sometimes so considered, but only an

TABLE V.—COMPARISON OF FIELD TESTS OF JOB CONCRETE.

Results obtained from different sources

Name of Work	Location	Period over which Tests were Taken	No. of Specimens, 28-day Tests Only	Class of Concrete, Field Tests: 28 days						Source of Data
				Proportions	Design St'gth, 28 days	fc	$\%fd$	V	V_1	
Building No. 10, Victor Talking Machine Co.	Camden, N. J.	11 wk. 10 wk.	187 80	1:2:4 1:1:2	2,000 3,000	2,190 3,930	110 131	16.6 13.8	80 77	W. A. Slater and S. Walker, *Proceedings*, A.S.C.E., Jan., 1925
Piers, Newark Bay Bridge, Central R. R. of N. J.	Newark, N. J.	10 wk. 6 wk.	225 39	1:2.4:3.6 1:2:3	3,000 3,000	3,150 3,800	105 127	11.9 11.5	78 79	do.
Reinforced concrete building, New York Telephone Co.	Brooklyn, N. Y.	5 wk.	76	1:5.5	1,650*	2,130	129	14.4	72	J. G. Ahlers and S. Walker, *Proceedings*, A.C.I., Vol. 20, 1924
Addition to Stadium, Polo Grounds	New York City	1 wk.	26	1:1.8:3.8	2,000*	2,390	117	8.2	100	do.
One-story addition to building, Ward Baking Co.	New York City	2 wk.	46	1:1.9:4.0 1:2.1:4.2	2,000*	2,180	109	11.7	83	do.
Warehouse, R. H. Macy & Co.	Long Island City, N. Y.	1 wk.	16	1:5	2,200*	2,260	103	14.7	62	do.
Foundations of building, New York Telephone Co.	New York City	2 wk.	15	1:4	2,400*	2,120	88	7.5	94	do.
Reinforced concrete building, Canada Cement Co., Ltd.	Montreal, Quebec	5 mo.	51 33	....	3,000 2,500	3,000 2,620	100 105	6.7 5.1	100 100	By letter, A. C. Tagge, Asst. Genl. Manager, Canada Cement Co., Montreal
Delaware River Bridge	Philadelphia, Pa.	....	107	1:2:4	2,000	2,730	137	...	84	A. W. Munsell, *Proceedings*, A.C.I., Vol. 21, 1925
Ross-Ade Stadium, Purdue University	Lafayette, Ind.	3 mo.	81	1:2:3	....	3,210†	...	11.3†	85†	W. K. Hatt, *Proceedings*, A.C.I., Vol. 21, 1925
Four reinforced concrete bridges, Bronx Parkway Commission	New York City	....	23 6 3	Various Various Various	3,000 2,500 1,500	3,510 2,670 1,520	117 109 101	9.7 18.2 19.5	91 83 67	W. F. Welsch, *Engineering News-Record*, Oct. 15, 1925
Combined results.....	....	......	32				114	15.7	88	
Seven highway bridges, California State Highway Commission	California	....	58	1:2:4	....	3,740†	...	10.3†	91†	H. D. Miller, *Engineering News-Record*, July 30, 1925
Stadium, University of Illinois	Urbana, Ill.	4 mo.†	27 150	1:2:4 1:2.5:3.33	....	1,550 2,400	... ...	14.5† 10.3†	71 77	W. A. Slater and R. L. Brown, *Proceedings*, A.C.I., Vol. 20, 1924

fc = Compressive strength, lb. per sq. in.
$\%fd$ = Compressive strength in per cent of design strength fd.
V = Mean variation in per cent.
V_1 = Number of tests falling between 80 and 120 per cent of average compressive strength, expressed as a percentage.

* Expected compressive strength at 28 days as given by formula $S = \frac{14000}{7x}$ where S = compressive strength at 28 days in lb per sq. in. and x = water-cement ratio (an exponent).

† These values obtained by scaling of published charts and not from tabulated lists of data, and consequently may be in slight error.

indication of the average strength of the concrete. If it approaches closely the strength for which the concrete was designed, then the basis for the design was correct, if not the design was incorrect. And by design strength is here meant the actual strength corresponding to the value of water-cement ratio used, which will vary with different materials and combinations of materials. When using the proportioning curves of Abrams, the field test should run several hundred pounds higher than the design strength, for these curves have been so chosen as to be safe under all but exceptional conditions and to do so they must of necessity be conservative. The commission's practice is to design for the actual compressive strengths called for, and this is only practicable because of the preliminary tests made and the special experience they possess. It is not recommended as a general practice, although where it is possible to follow it, it will prove more economical than the other system.

FIELD SUPERVISION AND REPORTS.

We have already referred to supervision and inspection. The part this plays in successful production of a quality concrete should not be overlooked. Connected with each job should be a man who has a sufficient knowledge of the properties of concrete to know the effect of the different malpractices to which it is subjected during manufacture, and who knows of what good workmanship consists and how to get it. Unless the concreting operations are in the hands of trained and competent men, the results of attempted control are almost certain to be disappointing.

In the commission's practice, great reliance is placed on the trained and competent man. He is usually an inspector who forms a part of the staff of the resident engineer. He is ordinarily one who has first received a year or more training in the laboratory and who has been picked for outside work because of his temperament and personality. This plan has been found more satisfactory than choosing likely men from the field, giving them a short course of training in the laboratory and then returning them to the field, for a man whose first experience is on construction, prides himself on being practical and never rids himself of the notion that testing is child's play and all book knowledge "high brow," whereas the man first trained in the laboratory has a different viewpoint and when he has obtained the practical experience of the former, makes a far safer and more valuable inspector. He knows the material he is working with, is accustomed to use tests as a means to knowledge and learns more from his experience.

A complete record of the concrete and concreting operations on a piece of construction is very desirable, particularly to an organization like the Hydro-Electric Power Commission of Ontario, which has to maintain and operate the structures it builds. It is almost impossible to have too detailed a record of the concrete but to get this may require an unreasonable amount of labor to prepare. Its compilation usually falls on the inspector, but the primary object of an inspector is to inspect, not to gather

statistics, and therefore complete records and complete inspection are antagonistic objectives.

This is a matter that has received a great deal of attention by the engineers of the commission, and they have adopted a practice midway between the extremes of complete records and no records. The inspector fills out a daily report sheet, requiring a minimum of clerical work in its compilation and designed to give a complete review of the concreting operations. Fig. 8 is an example of such a sheet. Besides this, the resident

Hydro-Electric Power Commission of Ontario

Report No. 125

JOB 'A'
POWER DEVELOPMENT

For Nov. 1 1925

Inspector's Daily Report of Concrete Materials

Materials		RECEIVED To-day	RECEIVED To Date	USED To-day	USED To Date	In Storage	Remarks
Cement	Bags	1760	25060	1818	23218	1842	
Gravel	Cu. Yds. Tons						
Fine Aggregate	~~Cu. Yds.~~ Tons	164	2780	215	2625	155	
Coarse Aggregate.	~~Cu. Yds.~~ Tons	50	4200	340	3925	275	
Steel Reinforcing	Tons		425		375	50	

Account No.	Location of Concrete	Elev.		Class	Cement Bags	Aggregate FINE ~~Cu. Ft.~~ Lbs.	Aggregate COARSE ~~Cu. Ft.~~ Lbs.	Water Gallons	W/C Ratio	Batches	Yards Cu. Sq.	Cyl. No.
	SUPPLY PIPE	340 TO	Concrete	A	6	1320	2100	22	.58	200	200	47
	UNIT No. 2	354	"	B	5	1320	2100	20	.65	120	120	48
			"									
			"									
			"									
			Grout		6	2400				3	2	

CYLINDERS BROKEN

No.	Age	Class	Lbs. Per Sq. In.
33	28	A	2750
33	28	A	2640
40	7	B	1060

ANALYSIS NOS.
Aggregate
Fine 79 Coarse 80

Per Cent. Moisture

Time	Per Cent.
FINE 9^{30} A.M.	3.2
COARSE 9^{30} A.M.	2.0
FINE 4^{00} P.M.	3.4

Time Started 7^{00} A.M.
Time Finished 7^{45} P.M.

WEATHER
Cloudy Snow
Showery ✓ Wind
Rain Fair

TEMPERATURE
Max. 56
Min. 34

Time of Concreting, Net 9^{00} HRS.
Time of Delays, Net 1.75 HRS.

No. of Mixers 1
Time of Mixing 1 MINUTE (NET)

John Smith Inspector

FIG. 8.—EXAMPLE OF DAILY CONCRETE REPORT USED BY HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO. AS FILLED OUT, THIS REPORT IS FOR A CONCRETING PLANT USING WEIGHING EQUIPMENT FOR MEASURING THE AGGREGATES.

engineer of each job prepares a statement that gives a complete description of plant and methods used for concreting, covering such details as the handling of the materials, placing and curing the concrete and in winter work details of any special precautions taken to protect it from frost.

The writer has found that this daily report, combined with a knowledge gained from one or more visits to the job, enables him to follow the progress of the concreting very closely, and to judge of the results being obtained. Also it furnishes a record of the work, which is sufficiently complete for purposes of reference at later times when for one reason or another the history of the concrete needs to be reviewed.

On a recent job the experiment was tried of putting a graphic wattmeter in the field laboratory and connecting it to the mixer motor. The chart obtained gave an indisputable record of the number of batches, of shutdowns and delays, of the time of commencing and finishing work and even of the mixing time of each batch and it was found to be a very useful addition to the records of the job and one which took almost no trouble to obtain.

CONCRETE MIXTURES SHOULD NOT BE DESIGNED FOR STRENGTH ALONE.

The purpose in designing mixtures is to insure the concrete having certain desired properties. Different conditions of service require concretes possessing different properties, as for instance compressive strength may be wanted, or watertightness or resistance to wear or weathering. A concrete may possess all of these properties to a high degree, but again, one or more may be almost entirely absent, and the presence of one is no guarantee of the presence of any other. Therefore, while it is necessary to rate a concrete by its strength, the design of a concrete mixture should not be based on strength alone unless strength is the only consideration. If other qualities are important, then the engineer should make sure that the proportions set will give the qualities desired and this means that in determining the proper mixtures, he must do more than blindly follow a set of rules or take his proportions from a table.

CONCLUSIONS.

Briefly, reference only is made to what the writer believes are the principal lessons to be learned from the seven years' experience of the Hydro-Electric Power Commission. These are:

1. The field control of concrete on the job is practicable and profitable.

2. The control of concrete on the job is applicable to work of all kinds and sizes.

3. The control of the quality of concrete is not a matter of designing mixtures nor of following some particular theory but it involves every step in the production of concrete from the selection of the materials to the curing of the product.

4. While the water-cement-ratio-strength relation is subject to variations, with average well-graded materials, no other means than this is necessary in designing concrete mixtures for a given compressive strength, but economy demands that cognizance be taken of the grading of the aggregates.

5. The concrete most economical in cement is not necessarily that having as much coarse aggregate as it is possible to use and still have its mixtures workable, but is usually one in which the sand is slightly in excess of this. The best combination of aggregate depends on the grading of both the fine and coarse aggregate.

6. Of the several processes in the manufacture of concrete, the measurement of aggregates and the placing of the mixtures are most important from the standpoint of quality.

7. Intelligent supervision of the materials and processes of concrete is essential to successful control of its quality.

8. The results of field tests of concrete should be interpreted with caution and in light of the conditions under which they were made, cured and tested, and for this season comparison of field tests from different jobs is difficult.

APPENDIX A.

The following case has been worked out in detail to illustrate the method followed by the Hydro-Electric Power Commission of Ontario in the calculation of the field proportions of concrete.

The conditions assumed are as follows:

Information	*Source*	1 cu. yd.
Capacity of mixer	Job plant	*Class A
Class of concrete	Engineering drawings	3.4 per cent by weight
Moisture in sand	Field tests	0.5 per cent by weight
Moisture in gravel	Field tests	
Weight of concrete	Proportioning data Table IV	152 lb. per cu. ft.
Ratio of fine to		40/60 by weight
coarse aggregate	do.	6 bags
Cement per cu. yd.	do.	0.68 by weight
Water-cement ratio	do.	

From the capacity of the mixer can be determined the weight of a wet batch of mixed concrete.

Capacity of mixer—27 cu. ft. of wet concrete.

Weight of batch = 27 x 152 = 4,100 lb.

This 4,100 is made up of:

6 bags of cement 6 x 86†	= 516	520 lb
Water 520 x 0.68	= 354	354 lb.
Aggregate 4,100 — (520 + 350) =		3230 lb.

Of this the aggregate is divided as follows:

40 per cent is sand 0.4 x 3,230 =	1290 lb.
60 per cent is gravel 0.6 x 3,230 =	1940 lb.

The weight of moisture must be determined and corrected for:

Weight of water in sand	3.4 x 1290 =	44 lb.
Weight of water in gravel	0.5 x 1940 =	10 lb.

The field weights therefore are:

Sand	1,290 + 40 =	1330 lb.
Gravel	1,940 + 10 =	1950 lb.
Water	354 — 54 =	300 lb.

*See Table II for specification of classes used by the Hydro-Electric Power Commission of Ontario.

†The Canadian bag is 87½ lb. gross. The weight 86 lb. is net, based on an allowance of ½ lb. for the sack and 1 lb. for cement left in the sack after emptying into charging hopper.

Hence the trial proportions if by weight are:

Cement	6 bags
Sand (Moist)	1330 lb.
Gravel (Moist)	1950 lb.
Water	30 gal. (Imp.)
	or 36 gal. (U.S.)

If by volume, the trial proportions are:

Assuming the weight per cu. ft. of the materials as measured at the mixer as being:

Gravel		110 lb.
Sand		80 lb.
Cement		6 bags
Sand (Moist)	1,330 ÷ 80 =	16.6 cu. ft.
Gravel (Moist)	1,950 ÷ 100 =	17.7 cu. ft.
Water		30 gal. (Imp.)
		or 36 gal. (U.S.)

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DISCUSSION.

ALDEN C. SENSEL.—What is the relation between the strength of the specimens which are cured under very favorable conditions and the strength of the concrete in the mass, that is which is actually delivered on the job? Mr. Sensel.

MR. YOUNG.—I am afraid I cannot answer that. We have not taken cores of the concrete on the job. Mr. Young.

MR. SENSEL.—How can you control the concrete in the mass, then, when you have varying atmospheric conditions, both as to temperature and otherwise? Mr. Sensel.

MR. YOUNG.—I do not hold the view that tests are to be taken as the measure of its quality when the concrete went in. You have to use your judgment afterwards. If you place it right and cure it right, if the inherent quality is in the concrete, then, that concrete too must be alright; you may have more strength, in some cases than in others, but your concrete is right. Mr. Young.

D. S. COLBURN.—My impression was that Mr. Young was making good laboratory specimens—that was his aim. I want to raise the same point this man brought out; what is the relationship between these cylinders? He claims that when he took them a long ways he got a big variation. What is the relationship between them and the concrete in the job? Mr. Colburn.

MR. YOUNG.—I think I have been misunderstood, because we consider the testing of the field laboratory specimens of secondary importance. The concrete is the thing, and if we are satisfied that the concrete is right, we do not care what the specimens show, particularly, but we do think that they are a measure, within reason. If you know it was right to start with and you have not abused it afterward, it is probable that the concrete is right in place. Mr. Young.

GEORGE W. HUTCHINSON.—We have had occasion to drill pavement slabs and compare the cores taken from these slabs with cylinders from the same batch of concrete. About 27 projects were investigated containing a variety of materials and mixes. The average strength of the 6 x 12-in. cylinder was about 60 to 70 per cent of the average strength of the corrected drilled core strength. I believe you are very well assured that the field concrete is stronger than that of the 6 x 12-in. cylinder as far as average strength is concerned. The trouble, to my mind, is that the variation from the average is a factor much bigger than the average strength. Mr. Hutchinson.

We encountered extremes in this work and in one case the drilled core was 600 per cent of the strength of the laboratory cylinder cast from the same batch; the average variation in the concrete is around 100 per cent to 15 per cent in the cast core. I believe Mr. Young has put the situation as well as it can be put. He says you are working entirely on

average strength and when you are in the field you must forget average strength other than designing it to be consistent with your design, and then regulate the variation in the field concrete by decreasing manipulation, and avoiding free water so that you can get it in place without segregation. Variation in general comes from segregation and improper proportioning and not from faults in design.

Mr. Walker. STANTON WALKER.—In the tests carried out by the Joint Committee to which Mr. Young referred, I think there are some data which have a bearing on the relation between laboratory specimens and the concrete from the job, which are generally overlooked because they form such a small part of the data reported. Nine slabs were cast on the job and cured under job conditions. From the same batches, cylinders were made and cured under laboratory conditions. Cores were drilled from the slabs having the same ratio of diameter to height as the cylinders. Almost identical results were obtained from the cores as from the cylinders. Also columns were cast which were sawed into prisms and cylinders of height equal to twice the diameter. Again, very similar results were obtained from the columns as from the cylinders. The conclusion which Mr. Slater and I arrived at from a study of those data were that, in general, tests of laboratory cylinders are a very accurate indication of the quality of the concrete on the job.

Mr. Cohen. A. B. COHEN.—We are overwhelmed with data on the determination of the properties of concrete in the laboratory. What we need now, and I think our science has developed considerably in the last few years toward a better concrete, is an answer to the question: "How is the concrete on the job and how is it acting under varying temperatures?" I think it is a very serious matter when we get to pouring concrete around 40 deg. temperature. We look at the concrete and it seems to be in excellent condition. The concrete around 40 deg. temperature will show a very low strength and there is that difficulty or that possibility of stripping the forms, taking down the forms before that concrete has attained its strength. I think we ought to direct our efforts toward finding out what strength we can expect under temperatures around 40 deg. when much concrete is being placed.

Mr. Smith. GEORGE A. SMITH.—What method did you use in determining the workability of your concrete?

Mr. Young. MR. YOUNG.—I would like to find somebody who could define and determine "workability." I cannot do it. If you get concrete into place without pockets, without segregation and it handles properly, it is workable. If you cannot, it is not workable. As far as the laboratory measure of workability is concerned, "there ain't no such animal," as yet. What we have in the flow table is an approximation of it. Ask a field man if three or four mixes of the same flowability but of different proportions are equally workable as far as placing is concerned, and he will probably say no.

A. E. LINDAU.—The great lesson I have learned from Mr. Young's paper is the fact that over a period of years they have tried to make a good concrete, gone about it in a logical and systematic way, and finally they succeeded by the simple process of getting a concrete that has a variation of 6 per cent in strength, as I understand it. In other words, they are approaching the goal of uniformity of concrete, which is one of the most important things we can handle and that we can solve. Mr. Lindau.

E. O. SWEETSER.—The curves showing a great variation in tests due to curing conditions in the laboratory have been known to us for many years, who are particularly concerned in making tests. I think we should carry away from this meeting the impression that tests are not the final answer, but that it is possible to make tests under conditions which will be reasonably representative of the truth about the concrete in place. Tests are becoming more and more numerous on the part of contractors scattered throughout the country. Many of them are obliged to ship samples some distance to have tests made. The laboratory at Washington University, St. Louis, receives shipments of samples from considerable distances by engineers and contractors who want to know what strength the concrete is showing. Uniformity in the handling of specimens is greatly to be desired, and I think the best information that can be obtained will be found in the pamphlet issued by the American Society for Testing Materials, copies of which have been distributed by the Portland Cement Association, with recommended standards of practice for taking specimens and shipping them. We have had samples of 6-in. cubes sent in in open crates badly damaged by handling. Such tests are practically worthless from that standpoint; also, 6-in. cubes should not be used when the 6 x 12-in. cylinder has become the accepted standard. I think probably Mr. Young is over modest when he thinks his curves show bad conditions. I do not think they show bad conditions at all; I am afraid, Mr. Young, that the jumps in those curves were to be expected under the conditions under which you had to work. Mr. Sweetser.

W. K. HATT.—There may be as much difference as 20 per cent between a specimen that has dried out and a specimen that still has the water in it. Prof. Hatt.

F. R. MCMILLAN.—I think Mr. Young has made the point very well, that after all, an important question in concrete construction is whether the concrete works properly into place in the structure. If there remains any doubt that Mr. Young has been getting the results he points out as most desirable, I suggest that you visit the power house at Queenston, with its 100,000 yd. of concrete, the most perfect concrete I have ever seen in such quantity. You cannot look at this structure without realizing that the concrete was not only uniform from day to day, but from month to month. The walls are everywhere smooth and dense and the edges and corners are sharp and neat. It is one of the prettiest pieces of concrete to be found anywhere and it is proof that Mr. Young's methods do work. Mr. McMillan.

Control of Concrete Mixtures on University of Pittsburgh Stadium.

By W. S. Hindman.*

Much has been said and written in the last few years on the design and control of concrete mixtures and any engineer or contractor, who is interested, can supply himself with good information on this subject by securing, with little effort, the written reports of valuable research and experiences of others. I think we can assume that we are now able to determine, with a reasonable degree of accuracy, the proportioning of available concrete aggregates in such a way as to produce concrete of the desired strength for the work in hand, but the application in the field is not so simple and the difficulties encountered there have tended to retard the general use of this knowledge.

It is not the writer's intention to dwell at length on these difficulties or to repeat much that has already been said, but to show how these problems were solved and worked with a reasonable degree of satisfaction in the construction of the University of Pittsburgh stadium.

In order to supply the university with an adequate athletic plant, of which it was badly in need, it was definitely decided in June, 1924, to proceed with the construction of a new stadium. The stadium was completed, or as much as was contemplated at that time constituting the entire lower deck, by Sept. 24, 1925, and was used for the football season.

After making a preliminary survey of all the available property in the Oakland district on which a stadium might be constructed, the present location was selected for the reason that it seemed to satisfy most of the requirements, namely: The desirability of being located on the university campus, non-interference with other university buildings, reasonable cost of construction due to land values and natural contour of the ground, and accessibility from the main arteries of transportation.

General Features—The stadium is located on the campus and is convenient to the main arteries of traffic through the Oakland district of Pittsburgh. A survey was made of the entire district which shows that it is possible to park about 14,000 automobiles on paved areas within a radius

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*Consulting Engineer, Columbus, Ohio.

of five-eighths of a mile from the stadium, with no parking permitted on streets adjacent to the stadium on account of interference with and danger to pedestrians.

A thorough study was made to determine what type of structure would best fit this location, give a pleasing effect from an architectural standpoint and accommodate as many athletic activities as possible. The closed or bowl type, where a portion of the hillside is excavated and the seats placed on the slope with the opposite or downhill side supported on framework, was selected as being most suitable for the site.

The site selected, being in the background of the group of fine buildings in the Oakland district, permitted the use of concrete for the exterior with

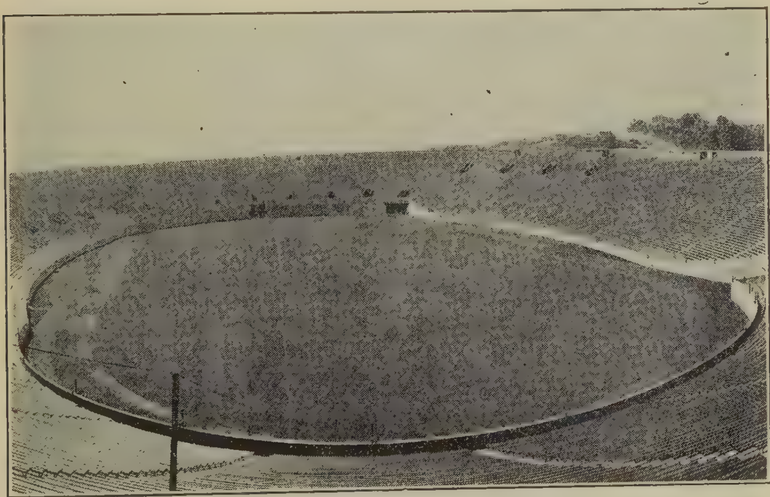

FIG. 1.—GENERAL VIEW OF UNIVERSITY OF PITTSBURGH STADIUM.

a simply architectural treatment. The so-called architectural embellishments are few, since we considered it as a massive structure and tried to give a pleasing effect to the whole by proper proportioning and placing of openings through the exterior wall. Only one entrance, that on the east side, is featured by using a large arch and projecting it out from the exterior wall. The entire architectural treatment is fitted to the interior layout so that there is very little lost space; thus giving an economic design.

The stadium is arranged to accommodate football, baseball, track, basketball and other indoor athletic activities besides being valuable for large public gatherings, pageants, etc. Provisions have been made for the installation of flood-lighting and amplifiers so that it can be used at night.

The structure at present consists of a single deck with approximately 65,000 permanent seats, but is designed so that a second tier of seats can

be added and the capacity increased by 30,000. All sight lines were carefully worked out and a good view of the entire area can be had from any seat. The structure is laid out on what is known as a three-centered oval or by using two radii. This makes all seat lines on a curve and all sight lines to converge toward the field, thus giving a greater equalization of seat values than if the sides were straight. In order to shorten the stadium on the long axis, we cut the quarter-mile track into the seat banks at the ends which eliminates the low undesirable seats at these points.

The main dimensions are: Overall length, 791 ft.; overall width, 617 ft.; length inside, 563 ft.; width inside, 343 ft.; height of lower deck above

FIG. 2.—MAIN ENTRANCE TO PITTSBURGH STADIUM.

playing field, 60 ft.; greatest height outside, 105 ft., which will be increased to 135 ft. when the second deck is added.

Construction Details—The interior circulation problem was very difficult to solve, since the entrances are from an elevation of 35 ft. below the field level to 60 ft. above. The structure is divided into 36 equal sections measured on the outside perimeter and each section is fed by means of an entrance through a portal, or from a doorway at the top of the seat bank. There are 21 sections fed directly from the street level and 15 by means of portal entrances from a large circulating or distribution gallery under the stands. There are five large entrances leading to this gallery. Two of these lead directly onto the gallery and it is reached from the other three

by means of large ramps. The system worked very satisfactorily and a capacity crowd was discharged from the stadium quickly without confusion.

The seats consist of three redwood strips screwed to malleable iron brackets which are attached to the concrete risers with galvanized bolts screwed into inserts which were placed when the concrete was poured. The box seats are individual folding chairs and each box holds eight or ten of these chairs.

In most stadiums previously built, little attention was given to supplying adequate toilet facilities, but this feature is well taken care of by installing twenty well-equipped and distributed public toilets not including those in the team quarters. Provision under the stands is made for both home

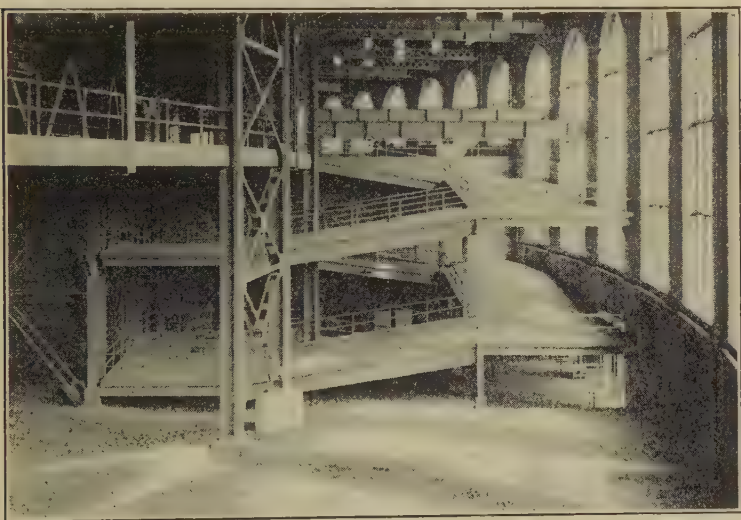

FIG. 3.—RAMP LEADING FROM CIRCULATING GALLERY TO EXIT.

and visiting team quarters, also a basketball room with a seating capacity of 4,500, which is steam-heated and provided with locker and shower rooms.

The stadium is built almost entirely of reinforced concrete and structural steel. Approximately 20,000 cu. yd. of concrete, 2,300 tons of structural steel, and 1,000 tons of reinforcing steel were used in the construction. The structural steel framing is entirely on the inside and is used to support about one-half the seat slabs. Structural steel was used under the high part to facilitate construction, since it could be erected in the winter time and is also slightly more economical than concrete framing. Approximately one-half the seats were poured directly on the slopes, but are designed as a flat slab supported on concrete piers so that in case the ground should settle, the structure will not be damaged. After construction had started,

it was discovered that almost the entire seat bank area resting on the slopes was undermined by old coal workings to a much greater extent than had been anticipated and was in very bad condition; also, we knew that a mine fire was burning under adjoining property, therefore it was decided to extend the supporting caissons or piers to a point on the rock underlying the coal seam. This was done and the columns are fire-proofed through the coal at the bottom by surrounding them with granulated furnace slag having a minimum thickness of 2 ft. All foundations were carried down to solid rock and the bottoms tested by drilling 4 ft. into the rock with an air drill.

Design of Concrete Mixtures.—We now come to the subject of concrete. For a number of years prior to the start of this project, the writer had

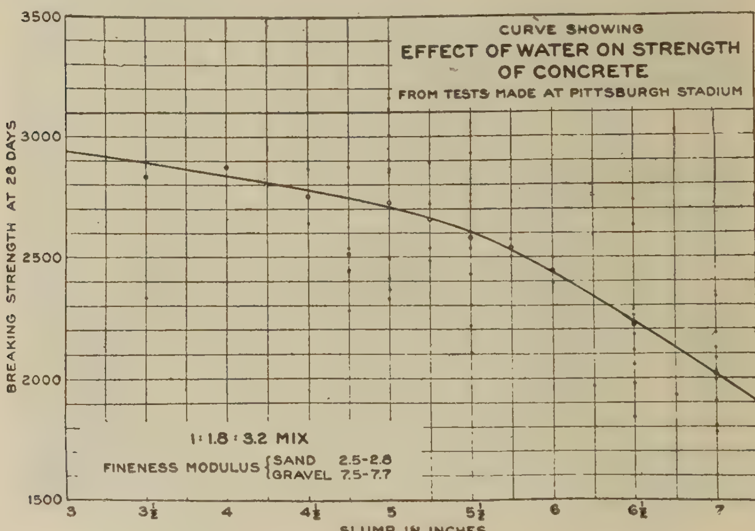

FIG. 4.—EFFECT OF WATER ON STRENGTH OF CONCRETE.

been interested in methods for securing better concrete, and was particularly anxious to get the best results possible on this stadium, since some similar structures have caused much trouble from poorly-proportioned, mixed and placed concrete.

In the early stages of development, we decided that it would be desirable to use two different strengths of concrete in the structure. Practically all the concrete in a stadium, except that in the foundations, is exposed to the elements, but the seat bank, or deck, is most severely tried on account of water penetrating the surface and freezing. Therefore, it was decided to use what is designated 2,500-lb. concrete for the deck and 2,000-lb. concrete for the balance. At this point, the writer realized the difficulty, at the present time, in trying to specify concrete mixtures by

strength thus leaving the contractors in the dark as to what proportions on which to base their estimates. In order to overcome this obstacle and determine on what aggregates to specify, we established in the basement of the house on the site in which the stadium office was located, a small experimental laboratory. In this laboratory, we tested out the designed mixes according to Abrams' theory, using various combinations of aggregates available in the Pittsburgh district. The fine aggregates consisted of river sand, sand manufactured by crushing coarse gravel, and slag screenings. The coarse aggregate consisted of river gravel and crushed slag with the fines screened out. From the results of these experiments, we were able to decide that the river sand and gravel offered the most economic solution and we also had a check on the probable mixture required to produce concrete in the 2,500-lb. class. It was decided to have all contractors, who figured on the superstructure, base their estimates on this mix. Therefore, the mix was indicated on the plans without classification, but with the understanding that the contract would call for designed mixes and that the owner would benefit by any saving in cement. This was agreed to by the Turner Construction Co., who were the low bidders, and the results will be discussed later in this article.

An interesting feature in producing concrete for this work was the required use of a sand inundator. It is a well-known fact that when 3 to 8 per cent of moisture is added to dry sand and then stirred up that a bulking or increase in volume of 25 to 30 per cent will occur, but that sand when thoroughly saturated with water settles down to almost the same volume as when dry. Therefore, the sand inundator invented by Mr. Escher, president of the White Construction Co., New York, and used by that company, overcomes much of the uncertainty in measuring sand.

The inundator is a watertight container with an adjustable bottom by means of which its capacity can be varied. A little more water than is required to saturate a batch of sand is placed in the inundator and the sand is then added from above through a shaking device until the inundator is filled. Water, in excess of the amount necessary to saturate the sand, overflows at the top. There is a uniform amount of sand and water in each batch. The variation of these ingredients in the usual methods of measuring concrete materials is responsible for many of our troubles in securing uniform workability and strength of concrete. The writer, after finding that the Blaw-Knox Co. were starting to manufacture inundators with a better mechanical device for operating them, specified its use on the work. The first one of these was installed, together with their batching hopper for measuring the coarse aggregate and gave us a mixing plant in which everything was controlled positively except for the variation of initial moisture in the coarse aggregate. The mixer was of 1-yd. capacity and bag cement was used.

Testing Methods—Practically all the concrete aggregates were tested before unloading from the barges. After acceptance they were delivered by trucks, a distance of approximately two miles, to the stadium site. It

was found after a few days' run that the sieve analysis, which was made each day, indicated only a slight variation in the fineness modulus of the sand and gravel, therefore we were able to establish a setting of the plant which seldom had to be changed for a certain class of concrete unless a marked fluctuation appeared in the materials. This accounts for some of the apparent variation in strengths of test cylinders, but the average strength is above that for which the mix was designed and considerable time in adjusting the mixing plant was saved.

In the beginning, it was decided to base the design of all our concrete mixes on a 4½- to 5-in. slump, as this consistency seemed to give about the proper workability for pouring the seat banks. Before the work had progressed far, it was found that, in order to speed up the work, it was necessary for the contractor to pour both classes of concrete in the same day from the one mixing plant. Therefore, at the request of the contractor who wanted to pour some of the walls in which a 2000-lb. concrete was called for, with a wetter mix, it was agreed that he be allowed to use the same mix as was figured for a 2500-lb. concrete but having a 6- to 7-in. slump. This required more cement than we had expected to use but speeded up the work, which was advantageous to both the contractor and owner. As has already been stated, enough cement had been estimated in the beginning for a 2,500-lb concrete throughout the job, therefore no additional cement over this amount was required by allowing the change mentioned above, but eliminated the contemplated saving in cement. The writer feels that the difficulties mentioned above are some of the most serious ones to overcome in attempting to place concrete in structures from a single mixing plant where limited quantities of different classes of concrete can be poured at one time. However, this would not apply to massive structures where the output of a mixing plant can be handled continuously for one class of concrete, provided the fineness modulus of the aggregates remains practically uniform.

The slump was determined on concrete, taken from the transporting car, immediately after it had been discharged from the mixer and as often as appeared necessary by the plant inspector or when there was an apparent change in the consistency. The standard prescribed method was used for determining the slump by filling a conical container, 8 in. in diameter at the base, 4 in. at the top, 12 in. high and rodding it with a ⅝-in. diameter rod. One thing of interest in connection with this is that concrete has a greater slump immediately after being discharged from the mixer than the same will have after being transported some distance to the forms and the workability at the point of deposit is affected accordingly.

Test cylinders were made from concrete immediately after being discharged from the mixer and after standing 24 hours were stored in damp sand until time to be sent to the laboratory for testing. During the months of April and May, the atmospheric temperature was low and as we had no means provided for heating the storage space, there was a noticeable drop in the strength of the cylinder due to this. As soon as the weather warmed up to about normal summer heat, there was no more trouble

from this source and the strengths checked out close to what was expected. The main things to provide for in field testing, if the results are to be depended on, are uniformity in making the specimens and provision for keeping the storage space at a uniform temperature of about 70 deg. F.

Concrete Handling and Conclusions—Practically all concrete for the superstructure was handled from a central mixing plant located 200 ft. from the stadium and at a point where the ground level was about the same elevation as the top of the seat bank. From this plant, a narrow-gage track was laid entirely around the top of the stadium and the concrete transported in 1-yd. hopper cars, drawn by gasoline locomotives, to a point nearest to where it was to be deposited. It was distributed from this point to the forms by means of chutes which it was not practicable to eliminate entirely but which were as short as possible and constructed so as to cause a minimum amount of separation of the concrete ingredients. All concrete materials were delivered in trucks to the site of the mixing plant where there was a large storage shed for cement and room for storing several thousand yards of sand and gravel. The bag cement was handled from this shed to the charging floor of the mixing plant by means of a belt conveyor and the aggregates to the bins over the mixer by means of a clam-shell bucket operated with a stiff-leg derrick.

Wooden forms were used for all concrete work and special attention was given to having the concrete thoroughly spaded after being deposited to insure a minimum amount of patching after the forms were removed. The plans and specifications called for all form boards to run vertically on exterior surfaces with a 4-in. horizontal band at about 10-ft. intervals. No work, with the exception of cleaning up and filling bolt holes, was supposed to be done on the concrete surfaces after the forms were removed.

In conclusion, the writer feels that the results obtained on this project from the use of designed concrete mixes and the sand inundator were very satisfactory and that any extra trouble or expense was justifiable. The accompanying chart, showing the relation of strength to slump or, in other words, the effect of water on the strength of concrete, is self-explanatory and indicates to what extent we were successful in obtaining uniformity in the concrete. One interesting feature is the apparent abrupt change in the direction of the curve near the point of the 5½-in. slump. The thing that appeals to us most, however, is the feeling that all concrete in the structure is good and that this is not entirely due to guesswork on the part of the man running the mixer. The general tendency at the present time is for engineers and contractors to co-operate in securing better concrete and it is to be hoped that eventually all who have to deal with this kind of work will thoroughly inform themselves as to the knowledge gained through research and apply it to their work. When that time comes, the number of concrete failures will be reduced to a minimum. At the present time, as is indicated in this paper, considerable judgment should be exercised in the application of modern methods or they might be made to work a hardship on the contractor.

The design and specifications for the stadium were prepared under the supervision of W. S. Hindman, engineer for the Stadium Committee, assisted by Miss Marion Hindman on architectural design. The construction was supervised by Stone & Webster, Inc., of Boston. The work was done by the Turner Construction Co., of New York, general contractors for the superstructure; the John F. Casey Co., Pittsburgh, grading and foundations; the New England Foundation Co., Boston, concrete caissons; the McClintic-Marshall Co., Pittsburgh, fabrication and erection of structural steel.

CHARLES E. NICHOLS.—The features of this work which particularly appeal to me are the comparison of the results in controlling the quality of the concrete with those of some jobs which have been analyzed in the past. The results as shown by the figures on the chart are not altogether encouraging in view of the considerable effort that was made to control the quality, but an analysis of the conditions which obtained, I think, satis- Mr. Nichols

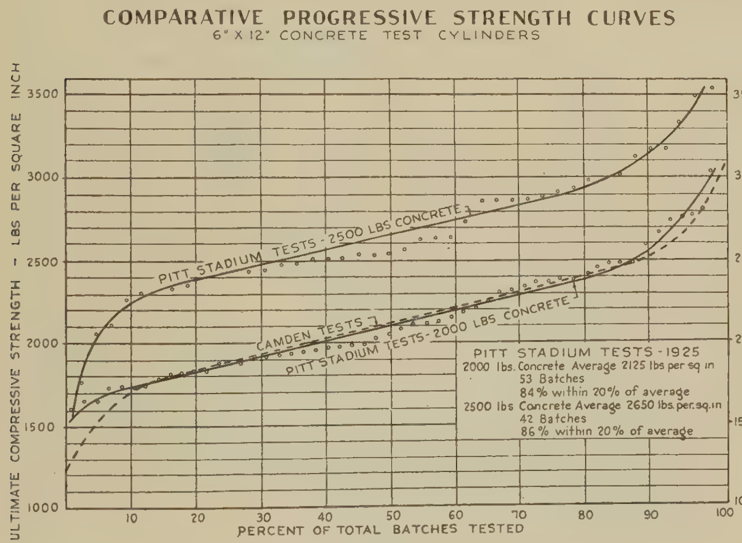

Fig. 1

factorily explains them. I have just two slides which I would like to show in this connection.

Those of you who have been here in past conventions have seen this form of curve (Fig. 1) which shows, at the left, the low strength shown by the cylinders, and then progressively the increases up to the high test. The dotted line shows the results of the tests on the Camden job which was perhaps the first job to have a consistent program of tests carried on and analyzed. Plotted in solid lines are the Pittsburgh stadium tests for 2,500-lb. concrete (above), and for 2,000-lb. concrete (below). Comparing the Pittsburgh stadium 2,000-lb. concrete curve with the Camden curve, you will note that they lie almost directly on each other. The Pittsburgh stadium tests did not run quite so low on the low values, and the general trend of the slope of the line is carried to a higher limit. As a matter of

fact 84 per cent of the total batches tested fell within 20 per cent below or 20 per cent above the average of all the batches. On the 2,500-lb. concrete, while the low and the high values were a little wider, the average was better; 86 per cent were within 20 per cent of the average.

On the face of it that would indicate that the Pittsburgh stadium concrete was more uniform in quality than the Camden concrete, and I think there are two general reasons for that. In Fig. 2 I have analyzed what might be called the accuracy of testing technique. For any given batch of three cylinders we have a batch average which is the average plotted on the curve of Fig. 1, and each individual cylinder deviates from

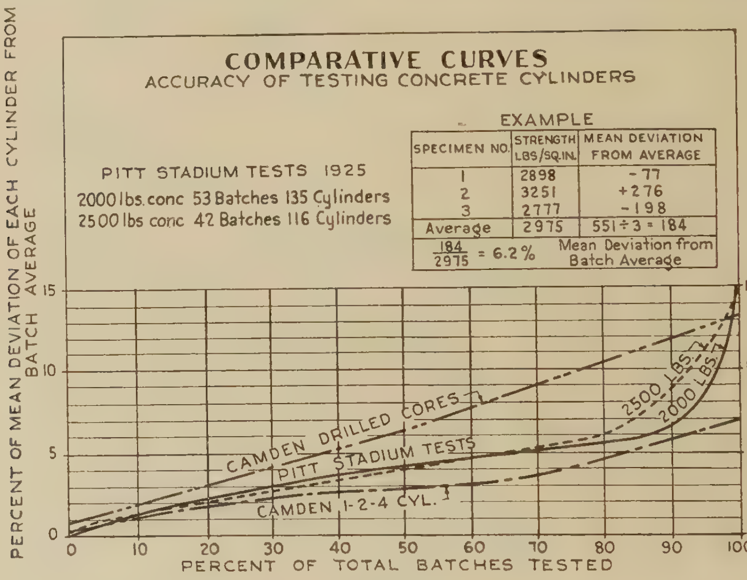

Fig. 2

that average a certain amount. The curve of the mean of deviation from the average of each batch is an indication of the accuracy, or lack of accuracy in making the tests.

Presumably the concrete for one batch is uniform and all three test cylinders should show the same strength. The fact that they do not indicates that the tests are not all conducted uniformly. In comparison with Camden again, plotting progressively the lowest mean deviations up to the highest, the Camden 2,000-lb. concrete tests show very little variation of any individual cylinder from its batch average. The Pittsburgh stadium tests which are plotted between the curves of the Camden cylinders and the Camden drilled cores, show a greater variation than the Camden test cylinders, because at Camden the testing organization was quite expert in

testing work, while on the Pittsburgh stadium job the organization was less experienced and greater variation was to be expected.

Working back then to the other curves, Fig. 1, we deduce, since the testing was not so accurate and since inaccuracy in the tests always results in lower, rather than in higher, values, that the Pitt stadium concrete actually was of better value than the tests indicated. If the Pitt stadium curves were corrected for that condition, noting that the correction of the curves would be greater for low values than for high values, we would have our curves flattening out approaching the horizontal, as well as rising bodily on the diagram. This would indicate then, in spite of the actual, almost coincidence of the 2,000-lb. curve with the Camden curve, that the Pittsburgh stadium concrete really was considerably more uniform in quality.

Another feature which perhaps tended to some greater variation in the tests was, as Mr. Hindman brought out just previously, that the strengths being obtained were all greater than were required by the design, and consequently the feature which was emphasized throughout the job was uniformity of consistency for any given part of the structure. We did not want a wet, sloppy batch at any point on the deck in the midst of a lot of nice dry concrete; we wanted it all uniform. Consequently, when there was some variation in the grading of the aggregate, sufficient to affect the consistency, they did not go to the trouble of changing the proportion of sand and gravel, which would have delayed work, but varied the water content, sacrificing uniform strength for uniform consistency. That procedure took care of variation not only in the aggregate grading but also in the moisture condition, the humidity on different days and different parts of the day, and gave the construction organization essentially a uniform consistency of concrete day in and day out which they could handle and place with remarkable economy and speed.

Use of the Water-Ratio Specification on the Portland Cement Association Building.

By F. R. McMillan* and Stanton Walker.†

Introduction.

The quality of concrete in the office and laboratory building of the Portland Cement Association was successfully controlled, without the use of special equipment and without the need of a testing organization, solely by means of a specification based on the control of the quantity of mixing water. This specification fixed only the proportion of water to cement; the aggregate proportions were left entirely to the judgment of the contractor, with the limitation that plastic and workable mixes be produced.

Most attempts to produce concrete of a predetermined quality during the past 6 or 7 years have been based on the control of the quantity of water by more or less minute regulation of the proportions, consistency, and grading of the aggregate, so that a given arbitrary mixture would give the strength desired. Some measure of success has attended these efforts, but it has developed that the methods are indirect and do not lend themselves readily to everyday production on the average job. Under these methods, questions of economy are confused with those of quality, and, as a consequence, unnecessary restrictions are imposed upon the selection of proportions and upon methods of transporting and placing the concrete. It was the recognition of the impracticability of these methods of control and the realization of the fundamental significance of the quantity of mixing water that led to the suggestion of the water-ratio specification.

The general law, developed by Abrams, that the strength of concrete is fixed by the water-ratio within the limitations that the mixes be plastic and workable and the aggregates clean and structurally sound, is so well established that no data need be presented here to support it. A study of published data[1] will show that this law has been widely accepted.

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[1]"Design of Concrete Mixtures," by Duff A. Abrams; Bull. 1, Structural Materials Research Lab., Lewis Institute. Chicago. See also other Bulletins of Laboratory, notably 4, 5, 11 and 13.

See Appendix II to Report of Committee C-9 on Concrete and Concrete Aggregates, Proc. Am. Soc. Testing Mat., 1922, p. 329.

"Wear and Compression Tests on Concrete," by R. B. Crepps; Proc. Am. Concrete Institute, 1920.

"Abrams' Water-Cement Ratio Checked in Germany," by V. P. Mann; *Engineering News-Record*, v. 94, p. 885, May 28, 1925.

"Water-Ratio Specification for Concrete," by F. R. McMillan and Stanton Walker; Presented before the Structural Div. of the Am. Soc. Civil Eng., Montreal, October, 1925.

"A New Test for Consistency of Paving Concrete," by F. H. Jackson and Geo. Werner; Proc. Am. Soc. Testing Mat., 1925, p. 204.

Opportunity is taken, however, to present for the first time some data from Prof. Abrams' tests which show the relationship between cement factor, workability and consistency. These data which will be found in Figs. 5 and 6, have an important bearing on the application of the water-ratio specification, as pointed out in the section "Further Discussion of Workability and Proportions."

It was one of the authors (McMillan) who first proposed the specification for concrete solely on the basis of the water-ratio and workability, leaving the consistency, proportions, and size and grading of the aggregates (which are purely questions of economy) to the contractor. It is worthy of record that the first specification of this type was that proposed by McMillan for the construction of the memorial stadium of the University of Minnesota. While this specification was not fully followed in the construction, it was for reasons wholly aside from its soundness or adaptability.

ORGANIZATION.

The design of the building and the supervision of the construction was in the hands of Holabird and Roche, architects, Chicago. The general contractor was the Turner Construction Co., New York. The caisson work was sublet to the Mid-Continent Construction Co., Chicago.

The architects were represented on the work by F. A. Winters; the Turner Construction Co. by H. B. Snell and C. H. Schwertner; and the Mid-Continent Construction Co. by M. H. Finley.

The specification for the concrete was written by the authors who kept in close contact with the construction to observe the results. Ralph Eckert acting under the direction of Mr. Walker, did all of the field testing and made the concrete cylinders for strength tests. The specimens were tested at the Structural Materials Research Laboratory, Lewis Institute, Chicago.

PRINCIPAL FEATURES OF THE SPECIFICATION.

The principal feature of this specification was the limitation of the maximum quantities of mixing water with little restriction on the proportions of cement to total aggregate or of fine to coarse aggregate other than that plastic and workable mixes be produced. Maximum slumps for the different portions of the work were stated to avoid any possibility of argument over unnecessarily wet mixtures or the accumulation of laitance. The usual clauses concerning quality of aggregates were included. Clauses fixing suitable limitations on the grading were also included. The maximum size of the coarse aggregate was required to be not greater than would permit of proper placement. The complete specification is appended.

TESTS AND TEST METHODS.

The operation of the specification was closely observed and a regular program of tests was carried out. The compressive strength of the con-

crete was determined at 3, 7 and 28 days in 6 x 12-in. cylinders taken each day from several batches immediately before they were deposited. Specimens were generally made for curing both in damp sand on the job, and in the moist room in the laboratory. The complete results of the strength tests are given in Tables IV and V. Summaries are given in Tables II and III, and in Figs. 1 to 3.

Frequent determinations of sieve analyses and of moisture content of the aggregates were made throughout the work. A summary of the sieve analyses is given in Table VI. The fineness modulus of each sample tested and the results of the moisture determinations are given in Table I. Fig. 2 shows the individual moisture determinations graphically. In general, the tests were made in accordance with the standard methods of the American Society for Testing Materials.

WATER-RATIO AND TOLERANCE.

Two classes of concrete were specified, as follows:

"Concrete for structural members shall be proportioned to give the necessary workability without exceeding the following ratios of water to cement:

"Where concrete of strength of 2,900 lb. per sq. in. is called for —6.0 U.S. gal. water per sack (94 lb.) of cement.

"Where concrete of strength of 2,000 lb. per sq. in. is called for —7.5 U.S. gal. water per sack (94 lb.) of cement."

These ratios were the maximum permissible, the operating tolerance covering unavoidable fluctuations was required to be entirely on the lower side so that the ratios specified would at no time be exceeded.

It was recognized that some tolerance in the water-ratios must be allowed, but to have specified a desirable mean value for the water-ratio with a fixed allowance by which this might be exceeded, assuming that there would be as many batches with a lesser value as with a higher value, would have offered no encouragement to more careful control, and might even have encouraged a variation all on the high side. Under the clause as written, it was to the contractor's advantage to keep the fluctuations at the lowest limit possible with the equipment used, for, by so doing, he was able to maintain an average water-ratio closer to the specified value and thus increase the amount of aggregates used with each sack of cement.

With this tolerance clause, the contractor was not restricted in the type of equipment for measuring or handling. Both the general contractor on the superstructure and the sub-contractor on the caissons used the simplest type of plant, which resulted in wider variations in water content than would have been the case with more accurate measuring devices. A study of the results of the strength tests show how this variation in the water-ratio affected the strength.

The complete results of the strength tests of concrete are given in Tables IV and V. Summaries in Tables II and III show averages for cer-

tain groups of tests, arranged in order of the mean temperatures on the days concrete was placed. Fig. 1 shows the strengths plotted for the different water-ratios. On this diagram also is plotted the curve repre-

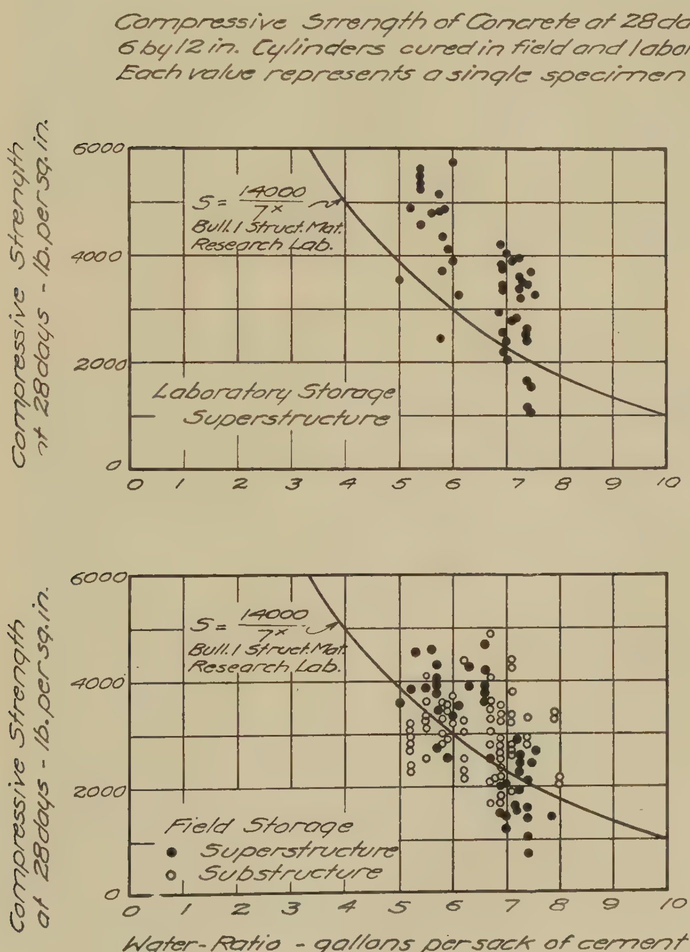

FIG. 1.—CONCRETE STRENGTHS FOR VARYING WATER RATIOS.

senting the water-ratio-strength law upon which this specification was based. This curve is that from Fig. 1, Bulletin 1, Structural Materials Research Laboratory, "Design of Concrete Mixtures," by Duff A. Abrams. In the equation shown on the diagram, S represents the compressive

strength at 28 days and x (an exponent), the water-ratio by volume. The equation has not been modified to show the relation with the water-ratio expressed in gallons per sack of cement.

The working out of the tolerance clause can be seen from Fig. 1. In the upper diagram of the figure, two groups of water-ratios for the 2,900- and 2,000-lb. concrete will be seen, averaging slightly over 5½ and 7 gal., respectively. In the lower diagram, about the same average grouping will be observed with somewhat more scattering of points. These averages are about ½ gal. below the specified limits and are indicative of the extra amount of cement required to avoid the outlay for more accurate equipment. It was necessary to set the water-measuring device so that with the probable variation in moisture in the aggregates and in the measurement of quantities, the specified water-ratios would not be exceeded.

WORKABILITY AND PROPORTIONS OF AGGREGATES.

The proportions of aggregates were governed solely by the requirements of workability, with the single limitation that the coarse aggregate should not be less than the fine, nor more than twice the fine. Workability was limited by the requirements that the mass could be puddled readily into the corners and angles of the forms, and around the reinforcement, without harshness or honeycombing and without the accumulation of water or laitance on the surface. Maximum slumps were specified for different portions of the building as follows:

Caissons	Max. slump 4 in.
Heavy walls, slabs and beams	" " 7 "
Thin walls and columns	" " 9 "

These liberal provisions gave the contractor the greatest possible latitude in the selection of proportions and consistency to get the necessary workability.

As it worked out there was no occasion for argument over the consistency or proportions, neither with the contractor for the caissons where the tendency was all towards the use of consistencies approaching the limit of stiffness, nor with the contractor on the superstructure where the tendency was all toward the opposite extreme. The old argument as to the amount of mixing water was entirely lacking. When it seemed desirable to have a more fluid mix, it was obtained by reducing the aggregate proportions, leaving the water and cement quantities in the batch unchanged. The conflicting desires on the part of the contractor always to have plastic workable mixes and yet to keep the cement factor low by the use of the largest possible amount of aggregates, resulted in mixes entirely within the range specified and very suitable for the work.

The average mixes used and the resultant slumps for the different classes of concrete and portions of the structure were as follows:

Class of Concrete 28-Day Strength lb. per sq. in.	Average Damp and Loose	Mix Dry Compact	Average Slump In.
	Substructure		
2,000	1 : 3.0 : 5.4	1 : 2.5 : 4.5	1½
2,900	1 : 2.4 : 3.5	1 : 2.0 : 3.25	1¼
	Superstructure		
2,000	1 : 3.0 : 3.9	1 : 2.5 : 3.6	7
2,900	1 : 2.0 : 3.0	1 : 1.7 : 2.7	6

MEASUREMENT OF MATERIALS.

The specifications required the contractor to measure the materials in such a way that the water-ratio could be closely controlled during the progress of the work and easily checked at any time. To avoid haphazard or unnecessary changes in consistency, the aggregates were required to be obtained from a source that would insure uniform quality and grading during each day's operation.

The aggregates were stored in small stockpiles on the street pavement. In the case of the substructure they were measured in wheelbarrows, and in buggies in the case of the superstructure. Frequent tests of the moisture content of the aggregates were made so that the water-measuring tank could be adjusted to supply enough additional water to give the correct water-ratios.

In the caisson work, the water was measured in a closed tank directly connected to the city water supply and equipped with a check valve which closed when the tank was filled. The tank was filled and discharged through a 3-way valve, the quantity discharged being regulated by means of a movable pipe. Owing to the inherent deficiencies in this style of tank, this device was not particularly satisfactory and it was necessary to make considerable allowance to insure that the specified water-ratios were not exceeded. The unreliability of this water-measuring device combined with the wheelbarrow measurement of aggregates, resulted in considerable lack of uniformity in the strengths, as will be noted in the lower diagram of Fig. 1.

For the superstructure, the water-measuring device consisted of a calibrated open tank provided with an adjustable overflow pipe which prevented more water being let into the tank than was required for the batch. This device permitted quite accurate measurements of the water, but corresponding accuracy was not obtained in the cases of the aggregates which were measured in buggies in fractional quantities. It was neces-

sary, therefore, to set the water container so that with the largest quantity of aggregates in any batch, the specified water-ratio was not exceeded. For those batches with smaller quantities of aggregate, a water-ratio below the specified amount was obtained due to the lesser amount of water carried into the batch through the medium of the moisture in the aggregate. This lower water-ratio did not show up in increased stiffness, but rather was accompanied by greater fluidity owing to the lesser amount of aggregate used. It did show up, however, in the strength tests, for a considerable part of the fluctuation in strengths shown in Fig. 1 and the tables was undoubtedly due to this change in water-ratio.

It will be of interest to consider the variations in results of strength tests in relation to the probable variations in the water-ratio arising from the inaccurate measurements of water and aggregates.

Of the 2,000-lb. per sq. in. range in strength shown in Fig. 1 for any water-ratio, about 500 lb. would seem to be chargeable to differences in temperature (see Tables II and III and Further Discussion of Strengths below). Of the 1,500-lb. per sq. in. variation remaining, about half would be a normal variation under well-controlled tests, leaving a difference of about 800 lb. per sq. in. to be accounted for by variations in water-ratio. This 800-lb. difference corresponds to a range of about ¾ gal. water per sack of cement or about ⅜ gal. either side of a mean value. A change of ⅜ gal. could be produced either by a change in moisture content of ¾ per cent in a batch of 4 cu. ft. of aggregate, or by an error in measurement of aggregate (assumed to be carrying about 4 per cent of moisture) of about 1 cu. ft. for each sack of cement. Such an error in aggregate measurement was not at all likely even under the method of measurement employed. However, when combined with small changes in moisture content, a sufficient error in measurement to give the ⅜-gal. variation was possible. From these considerations it seems that the variation in strengths beyond that normally expected is explainable by variations in water-ratio.

WATER CARRIED BY AGGREGATES.

The provision in the specification concerning the uniformity of grading of aggregates during each day's operations, would likely be important in many cases. Little difficulty was encountered in this work from variations in grading. The few occasions when the grading did vary showed the importance, from the contractor's point of view, of having uniform materials in order to avoid frequent changes in the proportions. From beginning to end of this work the change in proportions for any class of concrete was relatively small. This showed that the aggregates did not vary to an important extent from day to day, although the only requirement was that they be constant for each day.

The moisture content of the aggregate did not vary greatly, as will be seen from Fig. 2 and Table I. The greatest range between two successive determinations for the sand was 2.6 per cent; the usual range was less

than ½ per cent. Two-thirds of the values fell between 4 and 5 per cent and 90 per cent between 3.5 and 5.5 per cent of moisture. For the gravel, the variation was about the same.

The difficulties which may arise from variation in the amount of water carried by the aggregate have seemed to many a serious objection to the water-ratio specification. The experience on this building and other considerations indicate that under favorable conditions as to source of materials and facilities for storage, these difficulties may never be realized. Under the most unfavorable conditions the situation can be no worse than under other forms of specification with the advantage, however, that such control as is exercised under this specification is centered about the feature that is most important. This insures a better average

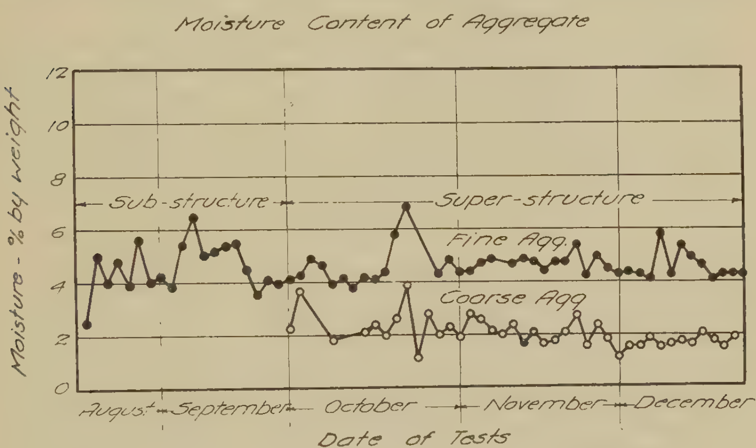

FIG. 2.—VARIANCE IN MOISTURE CONTENT OF AGGREGATES.

result than would be possible where control is centered about some feature, largely incidental to the production of concrete of definite strength, such as aggregate proportions.

Two methods of measuring the free water in the aggregates were used, one by weighing and drying, the other by displacement. The former was used principally as the equipment was already available. Some work was done in the development of convenient apparatus for determining moisture by displacement of water. With proper apparatus, this method should be somewhat simpler than drying and sufficiently accurate.

Quantity of Cement Used.

The question as to the probable effect of the water-ratio specification on the amount of cement required has been raised. On this point, the results from this building are most reassuring. In the caissons the average cement factor was 1.17 bbl. per cu. yd. for concrete showing a strength

of 2,880 lb. per sq. in. for an average of 87 specimens tested at 28 days. The contractor reported that this cement factor was about 1 sack per cu. yd. less than his usual practice for this class of work. This saving was obtained through the use of consistencies much stiffer than is the usual practice and comparatively large aggregate (maximum size 1½ in.). However, as stated previously the consistency was at no time too stiff for proper placement.

It is of interest to note that each caisson shaft, 70 to 75 feet in depth, was placed continuously in 3 to 4 hr. with no water appearing on the surface, although the surface was at all times plastic. During the half hour's time required to place the dowels at the top, a man standing in the concrete would gradually sink to a depth of 6 to 10 in. and small quantities of water would show in spots.

In the superstructure the average cement factor for the entire structural work including both classes of concrete was 1.47 bbl. per cu. yd. Of the total yardage, about 40 per cent was of the 6-gal. water-ratio and 60 per cent of the 7½-gal. water-ratio. The cement factor for 417 cu. yd. of concrete of the 7½-gal. water-ratio was 1.31 bbl. per cu. yd. This factor, as well as the average for the job, is slightly more favorable than the usual practice for a similar type of structure.

FURTHER CONSIDERATION OF STRENGTH DATA.

Brief reference has been made to the effect of temperature at time of placing concrete on the strengths shown in Fig. 1. A study of the averages in Tables II and III shows that while the data are not wholly consistent, there is a falling off of strength with the lower temperatures. This applies to the specimens cured in the moist room of the laboratory as well as to those cured under wet sand on the job. The inconsistency in these results is significant in that it indicates the important effect on strength of the treatment given to the specimen during the first 24 hr. Neither the recorded mean temperature for the day, nor even the extreme high and low, is necessarily a measure of the severity of exposure of the specimen. On this job it was frequently necessary to expose the freshly-made cylinders for several hours to low or near freezing temperature, while on other days they could be covered immediately after molding.

A general comparison of the cylinders cured in the laboratory and on the job can be had from Fig. 1; a more detailed comparison can be made from Tables II and III. In the discussion of the effect of variations in water-ratio on the strengths, it was assumed that differences in temperature would account for a variation of about 500 lb. per sq. in. A careful study of these data shows that this assumption was quite conservative; it is not unlikely that the difference was 700 or 800 lb., or even more.

During the cold weather, the tests at 3 and 7 days were studied carefully for indications as to the probable condition of the concrete in the structure. Generally it was felt that the strength of the concrete in the

structure was somewhere between that shown by the field and laboratory-cured specimen. In some cases, however, it was believed that it was even higher than the results shown by the latter where the temperature during the first few hours was particularly unfavorable. Examination of the

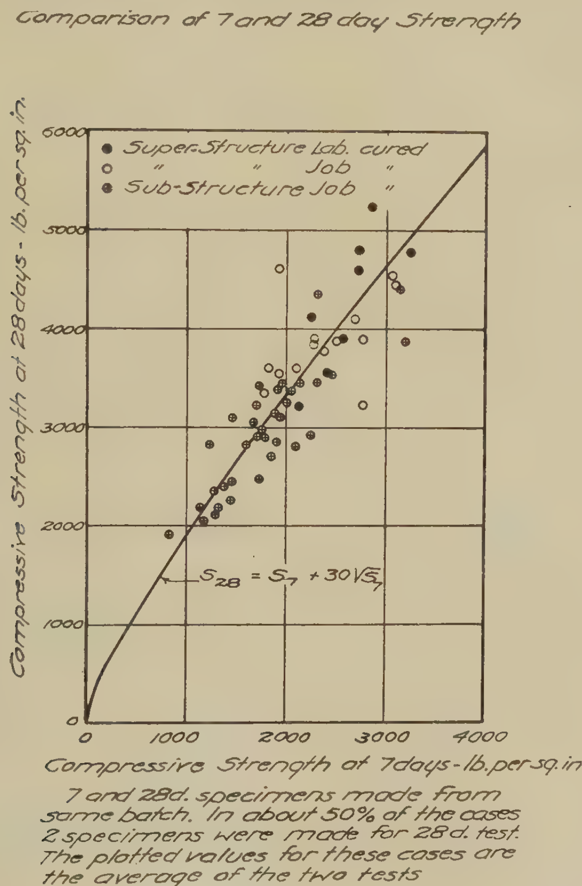

FIG. 3.—RELATION OF 7- TO 28-DAY STRENGTH.

structure itself corroborated this belief. At the time the forms were removed the concrete gave every indication of the high quality desired.

The numerical relation between the early strength and the 28-day strength is of interest, for if a general relationship can be shown, it will provide an accurate measure of the quality of the concrete at an early age. Fig. 3 shows the relation between the 7 and 28-day strengths for all tests where the specimens for both ages came from the same batch. The

curve is that given by Slater and Walker in the discussion of the tests at Camden and Newark Bay, sponsored by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete* and is based on the relation: 28-day strength equals the 7-day strength plus 30 times the

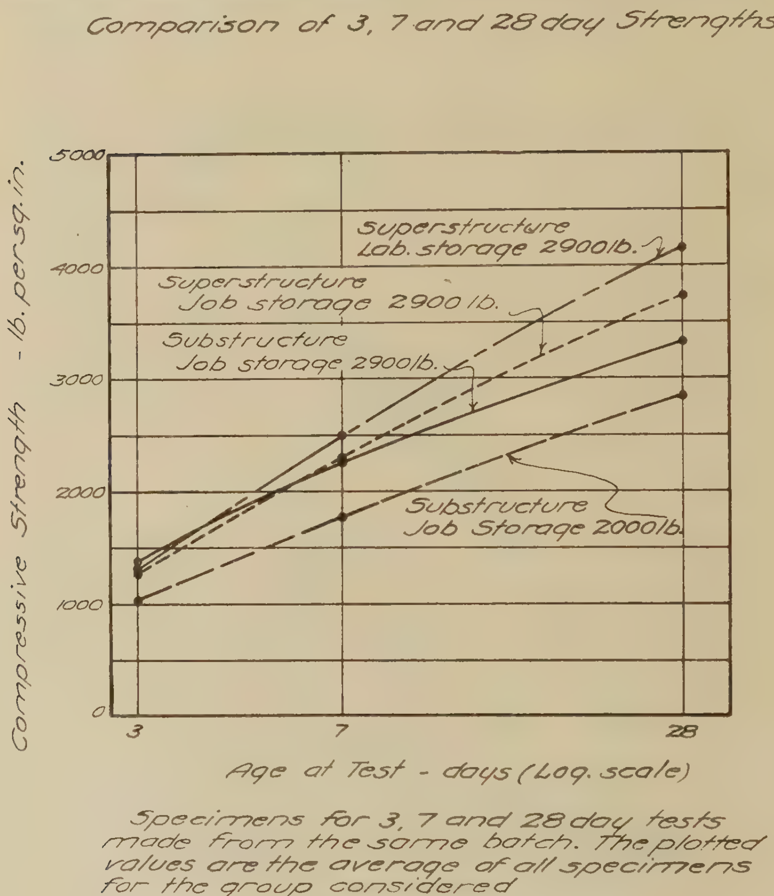

FIG. 4.—RELATION BETWEEN COMPRESSIVE STRENGTH AND AGE.

square root of the 7-day strength. It will be seen that this curve expresses quite accurately the mean of the plotted points for both the specimens cured in the laboratory and on the job.

Fig. 4 shows the relation between compressive strength and age for the 3, 7 and 28-day tests, the age being plotted to a logarithmic scale.

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*"Report of Field Tests of Concrete Used in Construction Work," by W. A. Slater and Stanton Walker; Proc. Am. Soc. C. E., January, 1925.

The plotted points are the averages of all of the specimens for the given conditions except that no specimen was included unless there were specimens for the other two ages from the same batch of concrete.

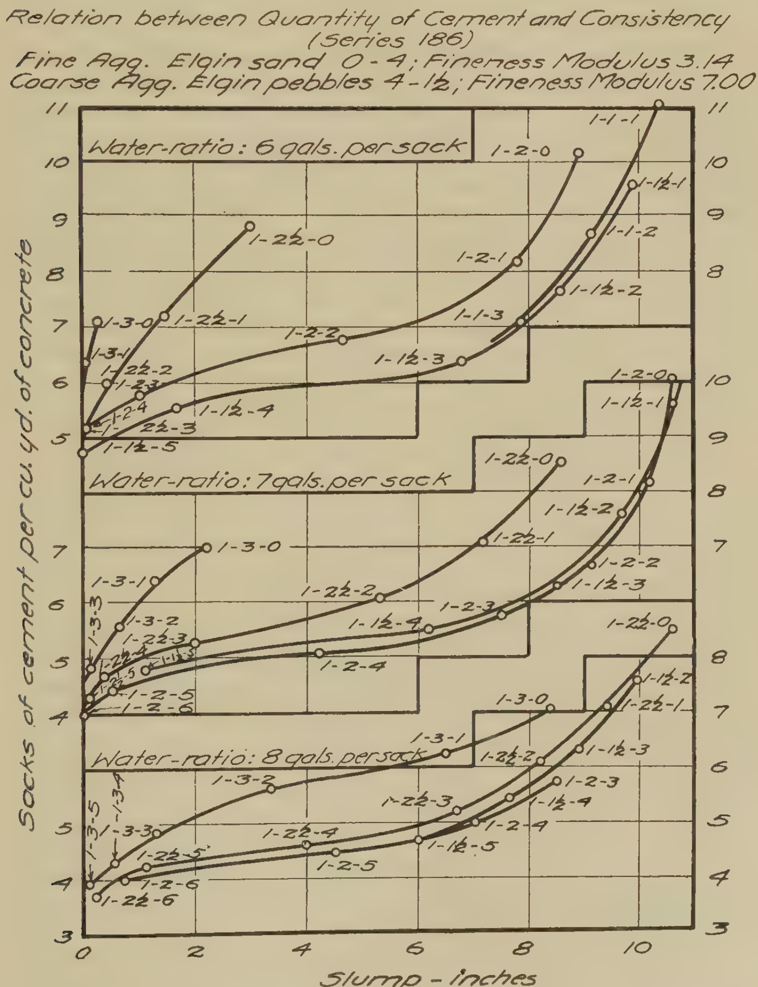


FIG. 5.—RELATION BETWEEN QUANTITY AND CONSISTENCY OF CEMENT.

The straight-line (or near straight line) relation between strength and logarithm of the age is characteristic of all the tests made at the Structural Materials Research Laboratory (See Fig. 14, Bulletin 7, Fig. 8, Bulletin 8, Fig. 6, Bulletin 12 and others).

FURTHER CONSIDERATION OF WORKABILITY AND PROPORTIONS.

Some engineers have expressed concern that conflicting desires concerning cement factor and cost of placing may not always be depended upon to give proper workability and that unsatisfactory work or at least a cause for argument may result.

In this connection, it will be of interest to consider some relationships between water-ratio, cement factor, and consistency which have been brought out by tests by Prof. Abrams made in 1923; Series 186—"Study of Workability of Concrete of Constant Water-Cement Ratio." The data from these studies, of interest here, are given in Figs. 5 and 6, which heretofore have not been published. Fig. 5 contains 3 diagrams for concrete made from sand and gravel aggregates for the 3 water-ratios; 6, 7 and 8 gal. per sack of cement. Each diagram consists of curves showing the relation between the sacks of cement per cubic yard of concrete and the slump for the given water-ratio. Each curve contains the results for a family of mixes; for example, the 1: 2 family consisting of mixes 1: 2: 0, 1: 2: 1, 1: 2: 2, etc., are all connected by a single curve. The mixes indicated are based on separated volumes of dry and rodded aggregates and are used in the usual sense, that is, 1 bag of cement, 2 cu. ft. of sand, 4 cu. ft. of coarse aggregate.

Attention is called to the fact that the curves in Fig. 5 are for this particular set of aggregates only. While other aggregates will show similar curves, the numerical relation between water-ratio, mix and slump will vary so widely with different aggregates that no attempt should be made to use these curves as a basis for estimating.

Many interesting relationships will be found from a study of these curves. The two points it is desired to make in this connection are first, the sudden break in the curves at about the 6-in. slump, and second, the rather well-defined limits of the proportion of fine to coarse aggregate within which workable mixes can be produced with a low cement factor.

As to the first point, the curves show that for a reduction in slump from 6 to 2 in. the saving in cement is slight, being less than ½ sack per cu. yd. for the mixes represented by the lower curves in each diagram. However, for slumps above 6 in., it will be seen that the extra cement for additional fluidity increases rapidly. For example, increasing the slump from 6 to 9 in. may require as much as 2½ sacks of cement per cu. yd.

To illustrate the second point, compare the mixes which will give a 6-in. slump using a water-ratio of 7 gal. per sack of cement. These are 1: 1.5: 4, 1: 2: 3.5 and 1: 2.5: 1.7, requiring respectively 5.2, 5.4 and 6.4 sacks of cement per cu. yd. For the usual range in prices the 1: 2: 3.5 mix will be found the lowest in cost for materials; it would also be more economical to place than the 1: 1.5: 4 mix because of the greater sand content. The 1: 2.5: 1.7 might be the lowest in cost to place, but in total cost the margin would be slight if any; however, because of the excess of fine material, this is an undesirable mix and would not be permitted under this specification.

Similar comparisons for other points on these curves will show that considerations of economy will almost always prevent the use of mixes that are too harsh, and to a considerable extent will prevent the use of mixes in which the proportion of coarse aggregate is undesirably low.

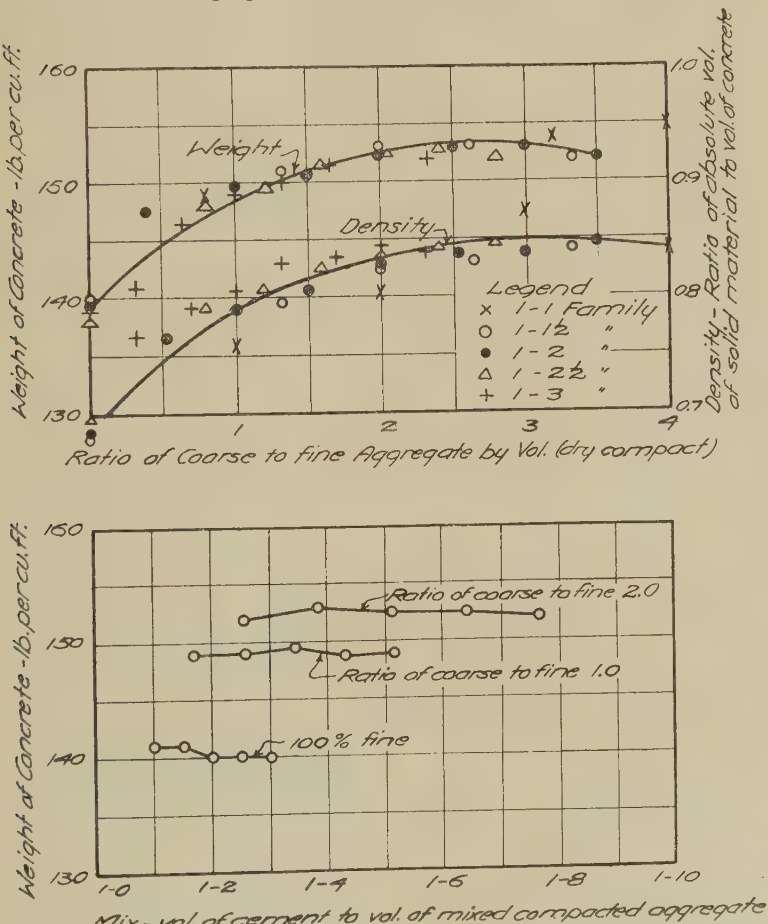


FIG. 6.—EFFECT OF VARYING PROPORTIONS OF FINE TO COARSE AGGREGATE ON WEIGHTS AND DENSITIES OF CONCRETE.

The tendency to use too little coarse can always be controlled by the specification of a suitable minimum ratio of coarse to fine.

In the specifications for the Portland Cement Association Building, the limits established were that the ratio of coarse to fine should not be less than 1 nor more than 2. It may be objected that this rather liberal margin is not in accord with the generally accepted notion that aggregate proportions should be adjusted to give the densest possible mix or the

largest possible proportion of coarse material consistent with proper workability. The authors of this paper both have been at different times and in different ways advocates of the use of large proportions of coarse material. However, the studies which led to the adoption of the water-ratio specification have pointed to the conclusion that, while it is true that denser mixtures, and, under certain very limited range of prices, more economical mixtures can be obtained by the use of larger proportions of coarse material, the gain in these respects is so slight as to make it undesirable to attempt such mixes at the risk of honeycombed structures or greater cost of placement.

In Fig. 6 will be found some data showing the effect of varying proportions of fine to coarse aggregate on the weights and densities of concrete made from sand and gravel. These data are based on Series 186 referred to previously, excluding those mixes which were greatly unworkable. From the upper diagram of this figure it will be noted that for proportions of coarse to fine, greater than 1, the changes in weight and density are relatively small. The difference of only 2 or 3 lb. per cu. ft. in weight which it is possible to obtain by the increase in the coarse material above this ratio does not compensate for the loss in workability and security against honeycombing.

For ratios of coarse to fine less than 1, it will be seen that both the weight and density fall off quite rapidly with decreasing ratio of coarse aggregate. It was for this reason that the minimum proportion of coarse to fine was placed at this value in the specification for the Portland Cement Association Building.

In considering Fig. 6, it must be remembered that the data are for a particular set of aggregates, 0 to 4 sand and 4 to 1½-in. gravel, typical of the materials around Chicago. Similar, though not identical curves will be found for other aggregates. The minimum ratio of coarse to fine should be reduced for smaller aggregates.

As pointed out previously there is very little likelihood of overharsh mixes being used because, with the usual range of prices, they do not prove economical even neglecting the extra cost of placing. Since there is little to be gained through the use of the mixes low in sand and much to be avoided, no objection can be raised to the specification of a maximum proportion of coarse aggregate.

The following table of limiting aggregate proportions is suggested for the ordinary range of materials to be used with a water-ratio specification, where it is required that the mixes be such that they can be

Size of Coarse Aggregate Inches	Ratio of Coarse to Fine on Basis of Dry Rodded Volumes	
	Minimum	Maximum
⅜	0.4	0.8
¾	0.7	1.5
1 and over	1.0	2.0

readily puddled into place without excessive spading and without harshness or honeycombing.

From the foregoing considerations it will be seen that under the water-ratio specification, the interests of the contractor and owner are more or less identical. The mixes, which the contractor finds economical, will avoid honeycombing on the one hand and excess water and its consequent porosity on the other.

LIMITATIONS OF THE SLUMP TEST.

In Fig. 5 and the discussion concerning it, use was made of the slump as a measure of consistency. In this use, it was not intended to recognize the slump test as an absolute measure of workability. In fact, the illustrations used are ample evidence of its limitations. Comparison was made of the 1 : 1½ : 4 and the 1 : 2 : 3½ mixes for the 6-in. slump and it was pointed out that the latter would be much more workable because of its greater sand content.

It is within the limits of truly workable mixes that the slump test is most useful. For example, if we were to compare the consistency of the 1 : 2 : 3 mix using 8 gal. of water to that of the same mix using 6 gal. of water, the slump of 8½ in. in one case and 1 in. in the other would give a very clear idea of the relative consistencies and in such a case the difference in workability. Similar comparisons could well be made between mixes in which the amount of cement and proportions of fine and coarse aggregates did not differ greatly; but to attempt to compare, on the basis of slump, the workability of mixes of wholly different proportions or of different kinds of aggregates is quite absurd.

Under conditions of uniform operation the slump test can be useful in indicating any change in the character of the materials, the proportions or the water content, as the test is very sensitive to such changes. It may not indicate which of these elements is the disturbing factor, but at least it shows the operator that some change has taken place and he can make the necessary modifications.

When the distinction between consistency and true workability is kept in mind the slump test is probably as good as any measure yet proposed for recording data such as in Fig. 5.

DATA FOR ESTIMATING QUANTITIES OF MATERIALS.

The objection is sometimes made that under the water-ratio specification, the contractor is not able to estimate accurately the amount of cement which will be required. The authors believe that the estimation of quantities should offer no serious difficulties. After a little experience with this specification the usual tables of quantities can be used; and in advance of such experience very reliable information can be obtained readily by making a few trial batches with the materials and of the consistency to be used.

It should be pointed out that with the water-ratio fixed, the most important factor affecting the quantity of cement is the consistency or

fluidity of the mix. This has been brought out in considerable detail in the discussion of workability, where the large increase in cement required to produce very fluid mixtures was illustrated from Fig. 5. In view of this dependence of the quantity of cement upon consistency, it is evident that the contractor must determine for himself what mix he will find necessary with the given materials to produce the consistency which he will require.

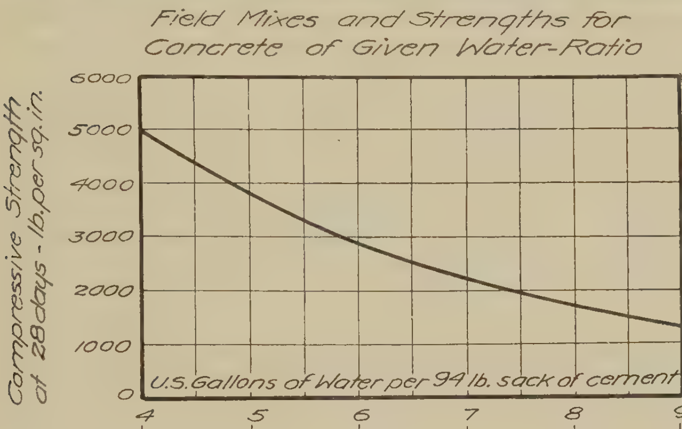
In the paper presented by the authors to the Structural Division of the Am. Soc. C. E., in October, 1925, a table was given which could be used as an aid to preliminary estimates of quantities of materials required. This table is reproduced in Fig. 7 with explanatory notes on the page facing. Attention is called to the fact that these data can be seriously misused. Evidence has already come to hand that the purpose and scope of this table have been misunderstood. In at least two cases it has been taken as the essence of the water-ratio specification and an attempt made to make portions of it the basis of a specification. It cannot be made too plain that the essence of the water-ratio specification consists only in the requirement of definite water-ratios with the single qualification that aggregates be used in such quantities and combination as will give workable mixes.

The water-ratio-strength relation shown in Fig. 7 is that from Abrams' "Design of Concrete Mixtures" and is the one adopted as the basis of the attached specification. When mixes which are plastic and workable are used with these water-ratios, the strengths indicated should be obtained with a fair degree of accuracy. Likewise, the quantities of cement shown for the different mixes, are quite accurate for materials of the same character and assumed condition as those used as the basis for the tables. These mixes are based on sand and gravel of the range of sizes and grading shown and on the assumption of a bulking of 25 per cent for the sand and 8 per cent for the coarse aggregate due to moisture and method of measurement. If the materials encountered in the field do not have the same percentage of bulking or are of different gradings or other characteristics, the cement quantities will not be the same for the indicated mixtures. The table is least accurate in the slumps given for the water-ratios and mixes indicated. Slight changes in gradings or surface characteristics may change the slump from one end of the range to the other. The best that is claimed for this feature of the table is that the data represent a fairly conservative estimate of the probable mixes required for average conditions to give the consistencies shown.

TESTING ORGANIZATION NOT REQUIRED.

The experience on this job demonstrated convincingly that no necessity existed for an experienced testing organization to control the proportions. This does not mean that the usual and necessary tests of quality of materials should be dispensed with. It is intended only to refer to the matter of controlling the proportions and consistency. The

determination of the proportions for each class of concrete was exceedingly simple, as it was done by trial with the first few batches. None of the computations usually considered a necessary part of scientific control of concrete was resorted to. Subsequent minor changes in proportions were made entirely by the concrete foreman. The water-measuring



Slump-in.	Mix (4)	Bbl. (4)	Mix (5)	Bbl. (5)	Mix (6)	Bbl. (6)	Mix (7)	Bbl. (7)	Mix (8)	Bbl. (8)	Mix (9)	Bbl. (9)
0 - No. 4 Aggregate; F.M. 2.75-3.25 (100% sand)												
½-1	1-2.0	3.1	1-2.8	2.5	1-3.5	2.1	1-4.1	1.8	1-4.7	1.6	1-5.2	1.4
2-3	1-1.6	3.5	1-2.4	2.7	1-3.1	2.2	1-3.7	1.9	1-4.3	1.7	1-4.8	1.5
6-8	1-1.2	4.2	1-2.0	3.1	1-2.6	2.5	1-3.2	2.1	1-3.8	1.8	1-4.4	1.6
0 - ⅜ in. Aggregate; F.M. 3.50-4.00 (65 to 85% sand)												
½-1	1-2.3	2.8	1-3.2	2.3	1-4.1	1.9	1-4.8	1.7	1-5.5	1.4	1-6.2	1.3
2-3	1-1.8	3.4	1-2.7	2.7	1-3.6	2.2	1-4.3	1.8	1-5.1	1.6	1-5.8	1.4
6-8	1-1.4	3.8	1-2.3	2.9	1-3.1	2.4	1-3.9	2.0	1-4.6	1.7	1-5.3	1.6
0 - ¾ in. Aggregate; F.M. 4.50-5.00 (45 to 65% sand)												
½-1	1-2.6	2.8	1-3.9	2.1	1-5.0	1.7	1-6.0	1.4	1-7.0	1.2	1-7.8	1.1
2-3	1-2.1	3.1	1-3.3	2.3	1-4.5	1.8	1-5.5	1.5	1-6.5	1.3	1-7.4	1.2
6-8	1-1.6	3.6	1-2.8	2.7	1-3.9	2.1	1-4.9	1.7	1-5.8	1.5	1-6.8	1.3
0 - 1½ in. Aggregate; F.M. 5.25-5.75 (30 to 45% sand)												
½-1	1-2.8	2.7	1-4.6	1.9	1-5.9	1.5	1-7.1	1.2	1-8.1	1.1	1-9.2	0.9
2-3	1-2.3	3.1	1-4.1	2.1	1-5.3	1.7	1-6.4	1.4	1-7.5	1.2	1-8.6	1.0
6-8	1-1.8	3.4	1-3.2	2.5	1-4.4	2.0	1-5.6	1.6	1-6.6	1.3	1-7.7	1.1
0 - 3 in. Aggregate; F.M. 6.00-6.50 (27 to 40% sand)												
½-1	1-3.0	2.5	1-5.3	1.6	1-6.8	1.3	1-8.1	1.1	1-9.3	0.9	1-10.3	0.8
2-3	1-2.5	2.8	1-4.6	1.8	1-5.9	1.4	1-7.2	1.2	1-8.4	1.0	1-9.4	0.9
6-8	1-2.0	3.2	1-3.5	2.2	1-4.9	1.7	1-6.0	1.4	1-7.2	1.2	1-8.3	1.0

From "Water-Ratio Spec. for Concrete" by McMillan & Walker, presented at the meeting of the Structural Div. Am. Soc. Civil Engrs. Oct. 1925 11-4-25

FIG. 7

device was set, with proper allowance for moisture, by the architect's representative. The only duties performed by the testing organization as a part of the control of the concrete, consisted in making the moisture determinations on the aggregates, which could have been done by any intelligent man.

EXPLANATION OF TABLE OF MIXES AND STRENGTH IN FIG. 7.

The principal purpose of this table is to aid in preliminary estimates of the quantities of materials required for concrete of a specified strength or water-ratio. The quantities, it will be seen, depend upon the size of aggregate and the consistency and may vary over a considerable range for concrete and any water-ratio.

Considerable confidence may be placed in the relation between strength and quantity of water shown in the diagram as this has been established by many tests. Also the quantities of cement for the different mixes will be found quite accurate for a wide range in type and grading of aggregate. However, considerable variations from the slumps shown may be expected due to minor differences in grading or type of aggregate. These can generally be overcome by adjustments in the proportions of fine to coarse or by slight changes in the total volume of the aggregate without sacrificing quality so long as workable mixtures and proper water-ratios are maintained.

Field mixes are in terms of total volume of separated aggregates measured damp and loose. These values were obtained by assuming the bulking of the sand, due to moisture and method of measurement to be 25 per cent and of the coarse aggregate 8 per cent. This method of indicating proportions should not be confused with the method of stating them in terms of volumes of dry and rodded mixed aggregates.

Water-ratio is expressed in U. S. gallons per 94-lb. sack of cement. The quantities of mixing water in the diagram (also at the heads of the columns in the table) include the moisture in the aggregates; the quantity to be added to the batch is this quantity less the amount carried by the aggregates.

Quantity of cement is in barrels per cubic yard to concrete and applies within the range of proportions of fine to coarse aggregates shown in the table.

Strengths for the different water-ratios are based on 28-day compression tests of 6 by 12-in. concrete cylinders puddled in the forms, cured in a damp place at normal temperatures and tested in a damp condition. In comparing the strengths of concrete made on the work, it is important that the tests be made in accordance with standard methods. Strengths are based on the use of portland cement meeting the minimum requirements of the standard specifications.

Aggregates on which the table is based were graded to a fair degree of uniformity between the sizes indicated. The division between the fine aggregate (sand) and coarse aggregate is the square mesh No. 4 sieve (approximately ¼ in.). The fineness modulus is the sum of the percentages in the sieve analysis, divided by 100, when the sieve analysis is expressed as percents coarser than the following sieves: No. 100, 48, 28, 14, 8, 4, ⅜ in., ¾ in., 1½ in., etc. An important characteristic of these sieves is that the size of the square opening of each is double that of the next smaller sieve.

Slumps indicated are for aggregates of rounded sand and gravel of the grading described and are based on tests using a frustum of a cone 12 in. high, top diameter 4 in. and bottom diameter 8 in.

SPECIFICATION FOR CONCRETE AND CONCRETE MATERIALS AS USED IN THE CONSTRUCTION OF THE PORTLAND CEMENT ASSOCIATION BUILDING.

Grand Avenue and Dearborn Street, Chicago.

SUGGESTIONS TO BIDDERS.

Attention to bidders is particularly called to the following concrete specification which differs materially from the ordinary form in that proportions of aggregates are not specified, while a limiting volume of water per sack of cement is definitely fixed.

This form of specification has been adopted because it has been definitely shown by ample tests and construction experience that it is the proportion of water to cement that determines the strength of the concrete. This means that so long as the ratio of water to cement remains the same, changing the amount of grading of aggregate does not affect the strength, but only the consistency of workability of the concrete; providing only that all mixes are plastic and workable and the aggregates clean and structurally sound.

Under this form of specification the contractor is permitted, within certain limits, to use such aggregates and proportions as in his judgment will produce proper economy and workability. By keeping the water-cement ratio constant, uniform strength is assured, while the necessary consistency is obtained by varying the aggregate proportions as desired. Increasing the quantity of aggregate for each unit of volume of cement, with its fixed ratio of water, increases the yield of concrete, but stiffens the mass and may increase the cost of placing. Increased plasticity can be obtained by reducing the amount of aggregates used with each sack of cement, or by changing the proportions of fine to coarse.

Bidders may find that in order to obtain the consistencies to which they have been accustomed, more cement will be required per cubic yard than was the case where increased fluidity was obtained by simply adding water. It is, therefore, suggested that trial mixtures be made to determine the cement requirements for the different consistencies which they will use in the various portions of the building. In such trial batches, the moisture contained in the aggregate must be taken into account in securing the water-cement ratio specified.

The following figures indicate how the cement requirement varies with the consistency. For concrete mixed with 6 U. S. gal. of water for each sack of cement, a consistency represented by a 6-in. slump would require 1¾ bbl. of cement per cu. yd. and by a 9-in. slump would require about 2¼ bbl. For concrete mixed with 7½ gal. of water for each sack of cement, consistencies represented by a slump of 4 in. would require about 1⅜ bbl. cement per cu. yd. and by a 7-in. slump about 1⅝ bbl. These figures are for average aggregates available in this market. Some variation will be found with different materials or with different gradings.

Attention is further called to the manner in which allowance must be made for batch-to-batch fluctuations in water content. Instead of specifying the average water-cement ratios desired with a definite tolerance, the maximum permissible values have been given, requiring the contractor to so conduct operations that these shall at no time be exceeded. This will enable the contractor to obtain increased economy as his methods of controlling the water are perfected, for, the smaller variation, the closer the maximum permissible limits can be approximated by the average water content.

The principal difficulties of controlling the mixtures under this type of specification will arise from variations in moisture content and grading of the aggregate, and from inaccuracies in measurement—principally with the sand. The requirement in the specifications, that the source of aggregate be such that uniformity of grading can be assured during any one day's operations, will largely eliminate some of the difficulty. This requirement will be found decidedly to the contractor's interest and one that in this market should offer no hardship. If methods of measuring aggregates are adopted which will give uniform quantities from batch to batch, not only will other difficulties be eliminated, but the annoyance of moisture variation in the sand will be considerably minimized. For, with uniformity in quantities of materials and grading, the correct moisture content in the concrete can be gaged with considerable accuracy by the workability or consistency as indicated by the slump test or by its appearance.

Determinations of the moisture content in the aggregate from time to time will be necessary in order that the water content will at all times be within the water-cement ratios specified. The frequency with which these determinations will be required and the consequent annoyance of making the requisite changes in the mix will depend entirely upon the care used in controlling the grading and uniformity of measurement and the moisture content in the aggregates.

The measurement of moisture in the aggregate within the limit of accuracy specified (2 lb. water in 100 lb. aggregate) should offer no great difficulty. Either drying and weighing a small sample or by inundation will give results satisfactory for the purpose.

Since this is likely to be a new form of specification to the bidder the owners will have on the job or immediately available at all times a man experienced in work of this character who will be ready to advise and assist the contractor in developing methods of carrying out the specifications and in overcoming any difficulties that may arise.

TABLE 1—MOISTURE CONTENT OF AGGREGATES

Date Made (1925)	Aggregate				
	Coarse		Fine		
	Fineness Modulus	Moisture, Per Cent by Weight	Fineness Modulus	Moisture, Per Cent by Weight	
		Drying		Drying	Inundation
	Substructure				
8-20	7.99	0	....	...	...
8-25	6.70	0	....	2.5	...
	....	...	....	...	4.0
	....	...	....	5.0	4.4
	....	...	3.02	4.0	...
	....	...	....	...	4.4
8-26	6.85	0	3.10	4.8	...
	....	...	....	3.9	...
	....	...	....	...	4.8
8-27	7.80	1.1	....	...	4.8
	....	...	....	5.6	...
8-31	....	...	3.11	4.0	6.0
9-2	....	...	3.05	4.2	4.4
9-5	....	...	....	...	3.6
	....	...	....	3.8	...
	....	...	....	5.4	...
	....	...	....	6.4	...
	....	...	....	5.0	...
	....	...	3.02	...	3.8
9-9	8.03	1.0	2.96	5.1	4.0
9-10	....	...	2.98	...	6.2
	....	...	....	5.3	5.6
9-11	7.98	0	2.91	5.4	4.2
9-14	7.92	0	2.90	...	4.0
9-16	7.89	0	3.10	4.4	3.2
9-17	7.99	0	3.02	3.5	...
9-19	....	...	2.96	4.0	...
9-21	7.75	1.3	3.20	3.9	...

Date Made (1925)	Aggregate				
	Coarse		Fine		
	Fineness Modulus	Moisture, Per Cent by Weight	Fineness Modulus	Moisture, Per Cent by Weight	
		Drying		Drying	Inundation
	Superstructure				
9-29	6.91	2.2	3.24	4.0	...
9-30	....	3.6	3.24	4.2	...
	....	...	....	4.8	...
10-1	7.05	...	3.41	4.6	...
10-2	....	1.7	3.12	3.9	...
	....	...	....	4.1	...
	....	...	....	3.7	...
10-3	7.00	2.0	3.06	4.1	...
10-5	6.75	2.3	3.09	4.0	...
10-7	6.93	1.9	3.18	4.3	...
10-8	6.48	2.5	3.23	5.7	...
10-9	6.76	3.8	3.23	6.8	...
10-10	7.00	1.0	....	...	3.4
10-12	7.10	2.7	3.36	...	2.0
	....	...	....	...	3.3
10-16	7.22	1.9	3.35	4.2	...
10-17	6.99	2.2	3.19	4.8	...
10-19	....	1.8	....	4.3	...
10-22	6.92	2.7	3.20	4.3	...
10-23	7.00	2.5	3.25	4.6	...
10-27	....	2.1	3.48	4.8	...
	....	2.0	....	...	...
10-29	....	2.3	....	4.6	...
10-30	6.96	1.6	3.49	4.8	...
10-31	....	2.0	....	4.7	...
11-3	....	1.6	....	4.4	...
11-11	6.97	1.7	3.43	4.7	...
11-12	7.13	2.0	3.39	4.7	...
11-13	6.89	2.7	3.43	5.3	...
11-14	6.97	1.5	....	4.2	...
11-16	6.96	2.3	3.43	4.9	...
11-23	7.29	1.8	3.31	4.4	...
11-25	....	1.1	....	4.2	...
	....	1.5	....	4.3	...
12-2	....	1.5	....	4.2	...
12-3	6.90	1.8	3.22	4.0	...
12-4	6.86	1.5	3.40	5.7	...
	....	1.6	....	4.2	...
12-5	6.62	1.7	3.43	5.3	...
12-8	....	1.6	....	4.9	...
12-9	7.01	2.0	3.47	4.6	...
12-14	....	1.7	3.18	4.0	...
12-15	....	1.5	3.47	4.2	...
1-5-26	6.92	1.8	3.18	4.2	...
1-6-26	6.91	1.8	3.15	4.1	...

TABLE 2—SUMMARY OF STRENGTH TESTS OF CONCRETE FROM CAISSONS

Compression tests of 6 x 12-in. cylinders.

Tests grouped according to atmospheric temperature at time of placing based on the mean temperature as shown by U. S. Weather Bureau Records for the days when concrete was placed.

Cylinders cured in damp sand on the job until a few days before test.

Mean Temperature, degrees F.	Age at Test, days	Specimens Cured on Job — Compressive Strength, lb. per sq. in. — Number of Specimens	Highest	Lowest	Average	Number of Specimens below 90 Per Cent of Required Strength
Required compressive strength: 2,000 lb. per sq. in. at 28 days. Water-ratio specified: 7½ gal. water per sack cement.						
81–85	3	4	1,190	820	1,080	
	7	4	1,880	1,230	1,640	
	28	32	4,950	1,730	3,010	1
76–80	3	9	1,440	320	940	
	7	9	2,100	810	1,590	
	28	20	3,570	1,710	2,720	1
71–75	3	6	1,300	830	1,030	
	7	6	1,920	1,300	1,620	
	28	12	3,120	1,710	2,730	1
64–70	3	5	1,770	760	1,270	
	7	7	3,140	1,130	1,890	
	28	16	4,440	2,110	2,930	None
55–60	3	3	1,450	1,190	1,280	
	7	3	1,880	1,410	1,710	
	28	6	2,890	2,340	2,650	None
Required compressive strength: 2,900 lb. per sq. in. at 28 days. Water-ratio specified: 6 gal. water per sack cement.						
81–85	3	..	.....	.....	.....	
	7	..	.....	.....	.....	
	28	6	4,360	3,320	3,780	None
76–80	3	1	1,450	.....	1,450	
	7	1	2,250	.....	2,250	
	28	4	3,300	2,600	2,930	1
71–75	3	1	1,230	.....	1,230	
	7	1	1,940	.....	1,940	
	28	2	3,570	3,360	3,410	None
65–70	3	4	1,840	690	1,380	
	7	6	3,180	1,470	2,200	
	28	12	4,400	2,920	3,540	None

March, 1926.

TABLE 3—SUMMARY OF STRENGTH TESTS OF CONCRETE FROM SUPERSTRUCTURE

Compression tests of 6 x 12-in. cylinders.
Tests grouped according to atmospheric temperature at the time of placing based on the mean temperature as shown by U. S. Weather Bureau Records for the days when concrete was placed.

Mean Temperature, degrees F.	Age at Test, days	Specimens Cured in Laboratory at 70° F.					Specimens Cured on Job				
		Number of Specimens	Compressive Strength, lb. per sq. in.			Number of Specimens below 90 Per Cent of Required Strength	Number of Specimens	Compressive Strength, lb. per sq. in.			Number of Specimens below 90 Per Cent of Required Strength
			Highest	Lowest	Average			Highest	Lowest	Average	
Required compressive strength: 2,000 lb. per sq. in. at 28 days. Water-ratio specified: 7½ gal. water per sack cement.											
46–50	3	6	920	480	650		5	730	260	560	
	7	6	1,630	790	1,110		6	1,180	450	860	
	28	6	4,020	1,660	2,640	1	6	2,570	1,180	1,790	4
41–45	3	1	750	.....	750		1	830	.....	830	
	7	1	2,200	.....	2,200		1	1,160	.....	1,160	
	28	1	3,860	.....	3,860	None	1	2,920	.....	2,920	None
36–40	3	11	1,110	420	630		11	890	340	540	
	7	11	2,080	1,170	1,500		11	1,540	660	1,030	
	28	11	4,610	2,200	3,590	None	11	2,620	1,480	2,150	1
31–35	3	8	830	590	700		8	800	480	600	
	7	9	1,780	1,050	1,330		9	1,130	530	860	
	28	9	3,460	2,480	2,830	None	9	2,780	1,080	1,820	5
24–30	3	4	940	180	490		4	880	230	540	
	7	3	1,080	530	770		3	740	460	640	
	28	7	2,970	1,010	2,580	3	7	2,420	740	1,770	3
Required compressive strength: 2,900 lb. per sq. in. at 28 days. Water-ratio specified: 6 gal. water per sack cement.											
61–65	3	..	.....	.....	.....		6	1,850	1,240	1,570	
	7	..	.....	.....	.....		8	3,100	2,290	2,710	
	28	..	.....	.....	.....	None	16	4,690	2,760	3,960	None
51–55	3	4	1,920	1,210	1,520		3	1,440	890	1,100	
	7	4	3,260	2,250	2,700		4	2,290	770	1,710	
	28	4	4,800	3,900	4,400	None	4	4,630	3,360	3,880	None
46–50	3	2	1,590	720	1,160		2	960	670	820	
	7	3	2,850	1,880	2,480		2	2,100	1,430	1,770	
	28	3	4,870	4,110	4,520	None	1	2,560	.....	2,560	1
41–45	3	8	1,260	510	1,030		2	1,190	670	930	
	7	2	2,140	1,970	2,060		2	1,920	1,490	1,710	
	28	6	5,740	3,220	5,110	None	4	4,580	3,550	4,420	None
36–40	3	2	1,020	500	760		2	790	420	610	
	7	1	2,440	.....	2,440		1	2,100	.....	2,100	
	28	1	3,540	.....	3,540	None	1	3,610	.....	3,610	None
31–35	3	1	1,380	.....	1,380		1	1,050	.....	1,050	
	7	1	2,260	.....	2,260		1	2,020	.....	2,020	
	28	1	3,720	.....	3,720	None	1	3,500	.....	3,500	None
24–30	3	4	1,060	410	850		4	1,260	350	860	
	7	2	2,950	2,850	2,900		2	2,060	1,920	1,990	
	28	4	5,120	2,400	4,180	1	4	3,860	1,380	3,050	1

March, 1926.

TABLE 4—TESTS OF CONCRETE FROM CAISSONS

Compression tests of 6 x 12-in. cylinders.

Tests grouped in order of date made. Temperatures are from the U. S. Weather Bureau Records for the days when concrete was placed.

Cylinders cured on the job in damp sand until a few days before test.

Date Made (1925)	Temperature degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Compressive Strength, lb. per sq. in. — Required Strength at 28 Days					
							2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean				3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
8-25	85	71	78	1:2.5-5	5.5	1	1,190	1,910	3,280	....	....	....
							....	....	3,570	....	....	....
				1:2.5-5	5.5	¾	920	1,690	3,390	....	....	....
							....	....	3,040	....	....	....
				1:2.5-5	5.5	1	950	1,590	2,560	....	....	....
							....	....	3,100	....	....	....
8-26	78	68	73	1:3:5	6.8	½	930	1,590	2,820	....	....	....
							....	....	2,780	....	....	....
				1:3:5	6.8	1½	950	1,680	3,070	....	....	....
							....	....	3,070	....	....	....
8-27	70	63	66	1:3:5	6.8	2	....	1,280	2,200	....	....	....
							....	....	2,520	....	....	....
				1:2.6:4.5	5.9	½	....	....	....	....	1,470	3,280
							....	....	....	....	....	2,920
				1:3.4:4.3	8.0	1½	....	1,130	2,240	....	....	....
							....	....	2,110	....	....	....
				1:2.6:4.3	5.9	½	....	....	....	....	2,320	4,400
							....	....	....	....	....	4,320
8-29	89	67	78	1:3:5	6.7	¾	1,410	1,990	3,070	....	....	....
							....	....	3,420	....	....	....
				1:3:5	6.7	1	630	810	1,710	....	....	....
							....	....	2,140	....	....	....
8-31	78	72	75	1:3:5	6.7	½	1,010	1,440	2,140	....	....	....
							....	....	2,780	....	....	....
				1:3:5	6.7	½	830	1,300	1,710	....	....	....
							....	....	2,520	....	....	....
				1:2.6:4.5	5.8	2	....	....	....	1,230	1,940	3,570
							....	....	....	....	....	3,360
9-2	92	71	82	1:3.4:4.3	6.9	3	....	....	2,850	....	....	....
							....	....	3,430	....	....	....
				1:3.4:4.3	6.9	1½	....	....	2,200	....	....	....
							....	....	2,240	....	....	....
				1:3.4:4.3	6.9	¾	....	....	2,310	....	....	....
							....	....	2,430	....	....	....
				1:3.4:4.3	6.9	2	....	....	2,560	....	....	....
							....	....	2,680	....	....	....
				1:3.4:4.3	6.9	1¼	....	....	1,810	....	....	....
							....	....	1,730	....	....	....
				1:3.4:4.3	6.9	1½	820	1,230	2,960	....	....	....
							....	....	2,710	....	....	....
				1:3.4:4.3	6.9	⅝	1,110	1,730	3,320	....	....	....
							....	....	3,540	....	....	....
				1:3:3.7	6.0	½	....	....	....	....	....	3,320
							....	....	....	....	....	3,320
				1:3:3.7	6.0	1½	....	....	....	....	....	3,640
							....	....	....	....	....	3,720
9-5	95	75	85	1:3.4:4.3	7.1	9	....	....	2,760	....	....	....
							....	....	2,740	....	....	....
				1:3.4:4.3	7.4	2⅜	1,190	1,880	2,960	....	....	....
							....	....	3,300	....	....	....
				1:3.4:4.3	7.4	1½	1,190	1,700	2,920	....	....	....
							....	....	2,850	....	....	....
				1:3.4:4.3	7.9	1½	....	....	3,390	....	....	....
							....	....	3,350	....	....	....
				1:3.4:4.3	6.7	1½	....	....	4,010	....	....	....

TABLE 4—TESTS OF CONCRETE FROM CAISSONS (*Continued*)

Date Made (1925)	Temperature degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Compressive Strength, lb. per sq. in. — Required Strength at 28 Days					
							2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean				3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
				1:3.4:4.3	6.7	1⅜	....	....	3,540	....	....	....
				1:3.4:4.3	6.7	1⅝	....	....	3,610	....	....	....
				1:3.4:4.3	6.7	1⅝	....	....	4,950	....	....	....
				1:3.4:4.3	6.7	1⅝	....	....	3,440	....	....	....
				1:3.4:4.3	6.7	1¾	....	....	3,680	....	....	....
				1:3.4:4.3	7.1	1⅞	....	....	3,860	....	....	....
				1:3.4:4.3	7.1	2⅜	....	....	2,820	....	....	....
				1:3.4:4.3	7.1	4	....	....	3,460	....	....	....
				1:3.4:4.3	7.1	2	....	....	1,920	....	....	....
				1:3:3.7	7.1	1¼	....	....	....	....	....	4,360
				1:3:3.7	7.1	1	....	....	....	....	....	4,290
9–9	91	69	80	1:3.4:4.3	6.9	1	840	1,330	2,170	....	....	....
							....	....	2,170	....	....	....
				1:3.4:4.3	6.9	2¼	790	1,170	2,170	....	....	....
							....	....	1,920	....	....	....
9–10	85	69	77	1:3.4:4.3	7.1	1	1,440	2,100	2,670	....	....	....
							....	....	2,960	....	....	....
				1:3.4:4.3	7.1	1⅞	....	....	3,360	....	....	....
							....	....	2,760	....	....	....
				1:2.7:3.3	7.1	1½	....	....	....	....	....	3,300
							....	....	....	....	....	2,600
9–11	73	68	70	1:3.4:4.3	5.8	2⅛	1,040	1,700	2,780	....	....	....
							....	....	3,030	....	....	....
				1:2.7:3.3	5.8	1⅞	....	....	....	1,620	2,310	3,520
							....	....	....	....	....	3,390
9–12	68	64	66	1:3.4:4.3	5.9	⅞	1,380	2,060	3,280	....	....	....
							....	....	3,480	....	....	....
				1:2.7:3.3	5.9	⅞	....	....	....	1,350	2,140	3,420
							....	....	....	....	....	3,480
9–14	68	62	65	1:3.4:4.3	6.2	1⅛	1,770	3,140	4,400	....	....	....
							....	....	4,440	....	....	....
				1:3.4:4.3	6.2	1	760	1,450	2,310	....	....	....
							....	....	2,230	....	....	....
				1:3.4:4.3	6.2	¾	....	....	2,510	....	....	....
							....	....	3,260	....	....	....
				1:3.4:4.3	6.2	1⅜	....	....	....	690	1,770	2,950
							....	....	....	....	....	3,010
9–16	68	63	66	1:3.4:4.3	5.5	⅞	1,410	2,460	3,600	....	....	....
							....	....	3,520	....	....	....
				1:2.4:3.0	5.5	1⅛	....	....	....	1,840	3,180	4,120
							....	....	....	....	....	3,640
9–17	86	64	75	1:3.4:4.3	5.2	1½	1,150	1,770	2,910	....	....	....
							....	....	2,840	....	....	....
				1:3.4:4.3	5.2	1¼	1,330	1,920	3,080	....	....	....
							....	....	3,120	....	....	....
9–19	91	69	80	1:3.4:4.3	5.2	¾	320	1,730	2,670	....	....	....
							....	....	2,280	....	....	....
				1:2.7:3.3	5.2	1½	....	....	....	1,450	2,250	3,140
							....	....	....	....	....	2,690
9–21	63	54	58	1:3.4:4.3	5.2	1½	1,190	1,880	2,890	....	....	....
							....	....	2,810	....	....	....
				1:3.4:4.3	5.2	3¼	1,450	1,840	2,640	....	....	....
							....	....	2,780	....	....	....
				1:3.4:4.3	5.2	1	1,200	1,410	2,460	....	....	....
							....	....	2,340	....	....	....

TABLE 5—TESTS OF CONCRETE FROM SUPERSTRUCTURE

Compression tests of 6 x 12-in. cylinders.
Tests grouped in order of date made. Temperatures recorded are taken from U. S. Weather Bureau Records for the days when concrete was placed.

Date Made (1925)	Temperature, degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Curing Condition	Compressive Strength, lb. per sq. in. — Required Strength at 28 Days					
								2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean					3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
9–29	69	63	66	1:2:3	5.7	2	Job	....	....	....	....	....	4,300
								....	....	....	....	....	4,080
				"	"	2½	"	....	....	....	1,560	2,780	3,725
								....	....	....	....	....	2,760
				"	"	3	"	....	....	....	1,240	2,530	3,820
								....	....	....	....	....	3,940
9–30	65	60	62	1:2.1:3.5	6.6	3¼	"	....	....	....	1,850	3,100	4,690
								....	....	....	....	....	4,220
				"	"	6	"	....	....	....	1,450	2,380	3,870
								....	....	....	....	....	3,760
								....	....	....	....	....	3,840
								....	....	....	....	....	3,600
10–1	70	60	65	"	6.3	2½	"	....	....	....	....	2,700	4,300
								....	....	....	....	....	3,920
10–2	67	59	63	1:2:3	5.3	2½	"	....	....	....	....	3,080	4,550
10–3	66	61	64	"	5.5	4½	"	....	....	....	1,540	2,290	3,870
								....	....	....	1,770	2,780	3,900
10–5	58	45	52	1:1.9:3	5.6	3	Lab.	....	....	....	1,650	3,260	4,770
				"	"	3¼	Job	....	....	....	....	1,970	4,630
10–7	52	44	48	1:1.8:2.5	5.4	7½	Lab.	....	....	....	....	2,710	4,580
10–8	59	43	51	1:1.9:3	6.0	6¾	Job	....	....	....	980	1,810	3,620
				"	"	"	Lab.	....	....	....	1,210	2,560	3,900
				1:2:3.5	"	7½	Job	....	....	....	890	1,770	3,360
				"	"	"	Lab.	....	....	....	1,370	2,250	4,120
10–9	46	37	42	1:1.9:3.5	6.1	7	Job	....	....	....	670	1,920	3,550
				"	"	"	Lab.	....	....	....	510	2,140	[illegible]
10–10	46	33	40	1:1.9:2.5	5.0	6¾	Job	....	....	....	790	2,100	3,610
				"	"	"	Lab.	....	....	....	1,020	2,440	3,540
10–12	56	49	52	1:2:3	5.74	5½	Job	....	....	....	1,440	2,290	3,900
				"	"	"	Lab.	....	....	....	1,920	2,740	4,800
10–16	55	43	49	"	5.82	7¼	Job	....	....	....	960	2,100	....
				"	"	"	Lab.	....	....	....	1,590	2,850	4,870
10–17	50	37	44	"	5.42	6½	"	....	....	....	1,230	....	5,340
				"	"	"	"	....	....	....	1,160	....	5,500
				"	"	"	"	....	....	....	1,020	....	5,200
				"	"	"	"	....	....	....	890	....	4,580
				1:2:3.2	5.73	6¼	"	....	....	....	1,090	....	5,230
				"	"	"	"	....	....	....	1,050	....	5,630
10–22	47	35	41	1:2:3	5.97	5	Job	....	....	....	1,190	....	....
				"	"	"	Lab.	....	....	....	1,260	....	....
				"	"	7½	Job	....	....	....	....	1,490	....
				"	"	"	Lab.	....	....	....	....	1,970	....
				"	"	8	Job	....	....	....	....	....	4,330
				"	"	"	Lab.	....	....	....	....	....	5,740
				1:2.67:4	7.44	3	Job	830	....	....	....	....	....
				"	"	"	Lab.	750	....	....	....	....	....
				1:2.67:3.3	7.07	6½	Job	....	1,160	....	....	....	....
				"	"	"	Lab.	....	2,200	....	....	....	....
				1:2.4:3.5	7.19	6	Job	....	....	2,930	....	....	....
				"	"	"	Lab.	....	....	3,860	....	....	....
10–23	54	40	47	"	6.84	"	Job	730	....	....	....	....	....
				"	"	"	Lab.	920	....	....	....	....	....
				"	"	3½	Job	....	1,080	....	....	....	....
				"	"	"	Lab.	....	1,630	....	....	....	....
				"	"	6½	Job	....	....	2,450	....	....	....
				"	"	"	Lab.	....	....	2,920	....	....	....

TABLE 5—TESTS OF CONCRETE FROM SUPERSTRUCTURE (*Continued*)

Date Made (1925)	Temperature, degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Curing Condition	Compressive Strength, lb. per sq. in. — Required Strength at 28 Days					
								2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean					3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
10–27	34	26	30	1:2:3	5.78	8	Job	....	....	....	350	....	....
				"	"	"	Lab.	....	....	....	410	....	....
				"	"	7½	Job	....	....	....	....	740	....
				"	"	"	Lab.	....	....	....	....	1,080	....
				"	"	8½	Job	....	....	....	....	....	1,380
				"	"	"	Lab.	....	....	....	....	....	2,480
				1:3.4:4	7.34	7	Job	230	....	....	....	....	....
				"	"	"	Lab.	180	....	....	....	....	....
				"	"	7¼	Job	....	740	....	....	....	....
				"	"	"	Lab.	....	700	....	....	....	....
				1:3.4:4.3	7.41	6½	Job	....	....	740	....	....	....
				"	"	"	Lab.	....	....	1,110	....	....	....
10–29	29	19	24	1:2:3	5.81	9	Job	....	....	....	1,260	....	....
				"	"	"	Lab.	....	....	....	980	....	....
				"	"	7	Job	....	....	....	....	1,880	....
				"	"	"	Lab.	....	....	....	....	....	....
				"	"	7½	Job	....	....	....	....	....	3,510
				"	"	"	Lab.	....	....	....	....	....	4,333
				1:3.4:3.7	7.29	6¾	Job	880	....	....	....	....	....
				"	"	"	Lab.	940	....	....	....	....	....
				1:3.4:4	7.35	7	Job	....	1,320	....	....	....	....
				"	"	"	Lab.	....	1,410	....	....	....	....
				1:3.3:3.7	7.22	8	Job	....	....	2,290	....	....	....
				"	"	"	Lab.	....	....	3,380	....	....	....
10–30	36	23	30	1:2:3.25	5.72	9	Job	....	....	....	850	....	....
				"	"	"	Lab.	....	....	....	960	....	....
				"	"	"	Job	....	....	....	....	1,920	....
				"	"	"	Lab.	....	....	....	....	2,950	....
				"	"	"	Job	....	....	....	....	....	3,460
				"	"	"	Lab.	....	....	....	....	....	5,120
10–31	43	28	36	"	5.78	6¾	Job	....	....	....	420	....	....
				"	"	"	Lab.	....	....	....	500	....	....
				1:2.9:4	6.10	8	Job	890	....	....	....	....	....
				"	"	"	Lab.	1,110	....	....	....	....	....
				1:2.7:4	6.89	6¾	Job	....	1,240	....	....	....	....
				"	"	"	Lab.	....	1,440	....	....	....	....
				"	"	"	Job	....	....	2,020	....	....	....
				"	"	"	Lab.	....	....	2,560	....	....	....
11–3	51	43	47	"	7.01	7	Job	260	....	....	....	....	....
				"	"	"	Lab.	550	....	....	....	....	....
				"	"	6⅞	Job	....	450	....	....	....	....
				"	"	"	Lab.	....	790	....	....	....	....
				"	"	7¼	Job	....	....	1,450	....	....	....
				"	"	"	Lab.	....	....	2,060	....	....	....
11–11	57	39	48	"	7.06	6	Job	670	....	....	....	....	....
				"	"	"	Lab.	550	....	....	....	....	....
				1:3.1:4	6.73	1⅞	Job	....	1,000	....	....	....	....
				"	"	"	Lab.	....	1,010	....	....	....	....
				"	7.01	2	Job	....	....	1,180	....	....	....
				"	"	"	Lab.	....	....	2,380	....	....	....
11–12	50	46	48	1:2.7:4	7.17	7½	Job	....	....	....	....	....	....
				"	"	"	Lab.	600	....	....	....	....	....
				"	"	7	Job	....	740	....	....	....	....
				"	"	"	Lab.	....	820	....	....	....	....
				"	"	7¼	Job	....	....	1,630	....	....	....
				"	"	"	Lab.	....	....	2,780	....	....	....
11–13	56	44	50	1:2:3	5.92	6½	Job	....	....	....	670	....	....
				"	"	"	Lab.	....	....	....	720	....	....
				"	"	6	Job	....	....	....	....	1,430	....
				"	"	"	Lab.	....	....	....	....	1,880	....
				"	"	7	Job	....	....	....	....	....	2,560
				"	"	"	Lab.	....	....	....	....	....	4,110

TABLE 5—TESTS OF CONCRETE FROM SUPERSTRUCTURE (*Continued*)

Date Made (1925)	Temperature, degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Curing Condition	Compressive Strength, lb. per sq. in.					
								Required Strength at 28 Days					
								2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean					3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
				1:2.9:4	7.84	7	Job	460	....	....	....	....	....
				"	"	"	Lab.	480	....	....	....	....	....
				"	"	7½	Job	....	720	....	....	....	....
				"	"	"	Lab.	....	910	....	....	....	....
				"	"	6¾	Job	....	....	1,490	....	....	....
				"	"	"	Lab.	....	....	1,660	....	....	....
11–14	45	36	40	1:2.7:4	6.89	7¼	Job	340	....	....	....	....	....
				"	"	"	Lab.	430	....	....	....	....	....
				"	"	6¾	Job	....	710	....	....	....	....
				"	"	"	Lab.	....	1,300	....	....	....	....
				"	"	6½	Job	....	....	1,480	....	....	....
				"	"	"	Lab.	....	....	2,200	....	....	....
11–16	37	32	34	1:2:2.9	5.86	9	Job	....	....	....	1,050	....	....
				"	"	"	Lab.	....	....	....	1,380	....	....
				"	"	"	Job	....	....	....	....	2,020	....
				"	"	"	Lab.	....	....	....	....	2,260	....
				"	"	8½	Job	....	....	....	....	....	3,500
				"	"	"	Lab.	....	....	....	....	....	3,720
				1:2.9:4.3	7.55	6½	Job	520	....	....	....	....	....
				"	"	"	Lab.	780	....	....	....	....	....
				"	"	6	Job	....	1,130	....	....	....	....
				"	"	"	Lab.	....	1,780	....	....	....	....
				"	"	8	Job	....	....	2,780	....	....	....
				"	"	"	Lab.	....	....	3,220	....	....	....
11–23	33	20	26	1:2.7:4	7.44	5½	Job	530	....	....	....	....	....
				"	"	"	Lab.	510	....	....	....	....	....
				"	"	6½	Job	....	710	....	....	....	....
				"	"	"	Lab.	....	1,080	....	....	....	....
				"	"	7	Job	....	....	1,310	....	....	....
				"	"	"	Lab.	....	....	1,520	....	....	....
				"	"	6½	Job	520	....	....	....	....	....
				"	"	"	Lab.	330	....	....	....	....	....
				"	"	5½	Job	....	460	....	....	....	....
				"	"	"	Job	....	530	....	....	....	....
				"	"	6	Job	....	....	1,010	....	....	....
				"	"	"	Lab.	....	....	1,010	....	....	....
11–25	46	34	40	"	7.27	5¼	Job	450	....	....	....	....	....
				"	"	"	Lab.	470	....	....	....	....	....
				"	"	6½	Job	....	790	....	....	....	....
				"	"	"	Lab.	....	1,170	....	....	....	....
				"	"	7	Job	....	....	2,200	....	....	....
				"	"	"	Lab.	....	....	3,500	....	....	....
				"	7.44	7½	Job	460	....	....	....	....	....
				"	"	"	Lab.	420	....	....	....	....	....
				"	"	7	Job	....	860	....	....	....	....
				"	"	"	Lab.	....	1,420	....	....	....	....
				"	"	"	Job	....	....	2,460	....	....	....
				"	"	"	Lab.	....	....	3,680	....	....	....
12–2	40	32	36	"	6.84	5½	Job	350	....	....	....	....	....
				"	"	"	Lab.	720	....	....	....	....	....
				"	"	"	Job	....	740	....	....	....	....
				"	"	"	Lab.	....	1,720	....	....	....	....
				"	"	"	Job	....	....	2,350	....	....	....
				"	"	"	Lab.	....	....	3,860	....	....	....
				"	"	7	Job	400	....	....	....	....	....
				"	"	"	Lab.	580	....	....	....	....	....
				"	"	"	Job	....	660	....	....	....	....
				"	"	"	Lab.	....	1,200	....	....	....	....
				"	"	"	Job	....	....	2,240	....	....	....
				"	"	"	Lab.	....	....	4,220	....	....	....

TABLE 5—TESTS OF CONCRETE FROM SUPERSTRUCTURE (*Continued*)

Date Made (1925)	Temperature, degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Curing Condition	Compressive Strength, lb. per sq. in.					
								Required Strength at 28 Days					
								2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean					3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
12-3	45	34	40	1:2.7:4	6.92	7	Job	780	....	....	....	....	....
				"	"	"	Lab.	670	....	....	....	....	....
				"	"	7	Job	....	1,080	....	....	....	....
				"	"	"	Lab.	....	1,450	....	....	....	....
				"	"	6¾	Job	....	....	2,020	....	....	....
				"	"	"	Lab.	....	....	3,320	....	....	....
				"	"	7	Job	660	....	....	....	....	....
				"	"	"	Lab.	780	....	....	....	....	....
				"	"	"	Job	450	....	....	....	....	....
				"	"	"	Lab.	620	....	....	....	....	....
				"	"	"	Job	....	1,380	....	....	....	....
				"	"	"	Lab.	....	1,620	....	....	....	....
				"	"	"	Job	....	1,350	....	....	....	....
				"	"	"	Lab.	....	1,810	....	....	....	....
				"	"	"	Job	....	....	1,950	....	....	....
				"	"	"	Lab.	....	....	3,800	....	....	....
				"	"	"	Job	....	....	1,990	....	....	....
				"	"	"	Lab.	....	....	3,460	....	....	....
12-4	51	42	46	"	6.98	6½	Job	690	....	....	....	....	....
				"	"	"	Lab.	790	....	....	....	....	....
				"	"	7½	Job	....	1,180	....	....	....	....
				"	"	"	Lab.	....	1,520	....	....	....	....
				"	"	6¾	Job	....	....	2,570	....	....	....
				"	"	"	Lab.	....	....	4,020	....	....	....
12-5	45	20	32	"	7.27	7½	Job	800	....	....	....	....	....
				"	"	"	Lab.	760	....	....	....	....	....
				"	"	"	Job	610	....	....	....	....	....
				"	"	"	Lab.	730	....	....	....	....	....
				"	"	8	Job	....	870	....	....	....	....
				"	"	"	Lab.	....	1,360	....	....	....	....
				"	"	"	Job	....	920	....	....	....	....
				"	"	"	Lab.	....	1,050	....	....	....	....
				"	"	6	Job	....	....	2,420	....	....	....
				"	"	"	Lab.	....	....	3,300	....	....	....
				"	"	"	Job	....	....	1,930	....	....	....
				"	"	"	Lab.	....	....	3,200	....	....	....
12-9	35	25	30	1:2:3	5.2	1½-2	Job	....	....	....	970	....	....
				"	"	"	Lab	.	....	....	1,060	....	....
				"	"	"	Job	....	....	....	....	2,060	....
				"	"	"	Lab.	....	....	....	....	2,850	....
				"	"	"	Job	....	....	....	....	....	3,860
				"	"	"	Lab.	....	....	....	....	....	4,870
12-14	36	27	32	1:2.7:4	7.21	7½	Job	480	....	....	....	....	....
				"	"	"	Lab.	610	....	....	....	....	....
				"	"	8	Job	....	820	....	....	....	....
				"	"	"	Lab.	....	1,260	....	....	....	....
				"	"	6¾	Job	....	....	1,560	....	....	....
				"	"	"	Lab.	....	....	2,810	....	....	....
				"	"	7	Job	....	650	....	....	....	....
				"	"	"	Lab.	....	1,380	....	....	....	....
				"	"	7¾	Job	....	....	1,520	....	....	....
				"	"	"	Lab.	....	....	2,720	....	....	....
12-15	33	29	31	"	7.41	8	Job	690	....	....	....	....	....
				"	"	"	Lab.	830	....	....	....	....	....
				"	"	7½	Job	....	950	....	....	....	....
				"	"	"	Lab.	....	1,550	....	....	....	....
				"	"	8	Job	....	....	1,550	....	....	....
				"	"	"	Lab.	....	....	2,620	....	....	....
				"	"	8½	Job	530	....	....	....	....	....
				"	"	"	Lab.	690	....	....	....	....	....
				"	"	7½	Job	....	950	....	....	....	....
				"	"	"	Lab.	....	1,070	....	....	....	....

TABLE 5—TESTS OF CONCRETE FROM SUPERSTRUCTURE (*Continued*)

Date Made (1925)	Temperature, degrees F.			Mix, Damp and Loose	Total Water, gal. per sack of cement	Slump, in.	Curing Condition	Compressive Strength, lb. per sq. in.					
								Required Strength at 28 Days					
								2,000 lb.			2,900 lb.		
	Maximum	Minimum	Mean					3 Days	7 Days	28 Days	3 Days	7 Days	28 Days
				1:2.7:4	7.41	8	Job	....	....	2,130	....	....	....
				"	"	"	Lab.	....	....	3,460	....	....	....
				"	"	7½	Job	550	....	....	....	....	....
				"	"	"	Lab.	590	....	....	....	....	....
				"	"	"	Job	630	....	....	....	....	....
				"	"	"	Lab.	620	....	....	....	....	....
				"	"	8	Job	....	530	....	....	....	....
				"	"	"	Lab.	....	1,160	....	....	....	....
				"	"	"	Job	....	950	....	....	....	....
				"	"	"	Lab.	....	1,370	....	....	....	....
				"	"	7¾	Job	....	....	1,420	....	....	....
				"	"	"	Lab.	....	....	2,480	....	....	....
				"	"	"	Job	....	....	1,080	....	....	....
				"	"	"	Lab.	....	....	2,520	....	....	....
1-5-1926	36	40	38	"	7.28	7½	Job	470	....	....	....	....	....
				"	"	"	Lab.	490	....	....	....	....	....
				"	"	"	Job	....	1,020	....	....	....	....
				"	"	"	Lab.	....	1,290	....	....	....	....
				"	"	8	Job	....	....	2,620	....	....	....
				"	"	"	Lab.	....	....	4,610	....	....	....
				"	"	"	Job	680	....	....	....	....	....
				"	"	"	Lab.	660	....	....	....	....	....
				"	"	7½	Job	....	1,540	....	....	....	....
				"	"	"	Lab.	....	2,080	....	....	....	....
				"	"	7	Job	....	....	2,280	....	....	....
				"	"	"	Lab.	....	....	4,330	....	....	....
1-6-1926	35	26	30	"	7.07	8	Job	1,380	....	....	....	....	....
				"	"	"	Lab.	1,020	....	....	....	....	....
				"	7.23	6	Job	....	2,000	....	....	....	....
				"	"	"	Lab.	....	2,580	....	....	....	....
				"	"	5½	Job	....	....	2,420	....	....	....
				"	"	"	Lab.	....	....	3,970	....	....	....
				"	"	6½	Job	1,160	....	....	....	....	....
				"	"	"	Lab.	1,280	....	....	....	....	....
				"	"	7½	Job	....	1,630	....	....	....	....
				"	"	"	Lab.	....	2,420	....	....	....	....
				"	"	"	Job	....	....	2,320	....	....	....
				"	"	"	Lab.	....	....	3,460	....	....	....
				"	"	8	Job	1,010	....	....	....	....	....
				"	"	"	Lab.	1,270	....	....	....	....	....
				"	"	"	Job	....	1,730	....	....	....	....
				"	"	"	Lab.	....	2,240	....	....	....	....
				"	"	7½	Job	....	....	2,310	....	....	....
				"	"	"	Lab.	....	....	3,600	....	....	....

TABLE 6—SUMMARY OF SIEVE ANALYSES OF AGGREGATES

Sieve analyses made with square mesh wire cloth sieves.

Aggregate	Amount Coarser than Each Sieve, Per Cent by Weight										Fineness Modulus
	Sieve Size										
	No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜ inch	¾ inch	1½ inch	3 inch	
Substructure											
Crushed Limestone:											
Highest..........	100	100	100	100	100	100	100	99	4	0	8.03
Lowest..........	100	100	100	100	100	100	70	0	0	0	6.70
Average (10 tests)	100	100	100	100	100	100	94	75	0	0	7.69
Sand:											
Highest..........	98	89	65	46	27	9	0	..	..	..	3.34
Lowest..........	99	88	50	30	17	6	0	..	..	..	2.90
Average (14 tests)	99	91	56	34	19	7	0	..	..	..	3.06
Superstructure											
Gravel:											
Highest..........	100	100	100	100	100	97	82	50	0	..	7.29
Lowest..........	100	100	100	100	100	89	45	14	0	..	6.48
Average (26 tests)	100	100	100	100	100	95	68	31	0	..	6.94
Sand:											
Highest..........	98	93	70	48	30	10	0	..	..	..	3.49
Lowest..........	99	93	57	32	18	7	0	..	..	..	3.06
Average (29 tests)	99	91	63	42	25	10	0	..	..	..	3.30

March, 1926.

SPECIFICATIONS FOR CONCRETE AND CONCRETE MATERIALS.

WATER-CEMENT RATIO.

Concrete for structural members shall be proportioned to give the necessary workability without exceeding the following ratios of water to cement:

Where concrete of strength of 2,900 lb. per sq. in. is called for
—6.0 U. S. gal. water per sack (94 lb.) of cement.

Where concrete of strength of 2,000 lb. per sq. in. is called for
—7.5 U. S. gal. water per sack (94 lb.) of cement.

These water-cement ratios are the maximum permissible. The mixes shall be proportioned for somewhat lower ratios so that with the normal fluctuations which may be expected from batch to batch these ratios will not be exceeded. Water or moisture contained in the aggregate must be included in computing the water-cement ratios. Water absorbed by the aggregate in a period of 30 minutes may be deducted in computing the water-cement ratio.

The water-cement ratios specified shall not be changed except by the architect. In the event that the architect finds it necessary to change the water-cement ratios from those specified, adjustment, covering amount of cement and aggregates affected, will be made as an extra or a credit under the provisions of the contract.

MEASURING MOISTURE IN THE AGGREGATE.

Moisture in the aggregate shall be measured by a method, satisfactory to the architect, which will give results within 2 lb. for each 100 lb. of aggregate.

CONCRETE PROPORTIONS AND CONSISTENCY.

The proportions of aggregate to cement for concrete of the water-cement ratios specified shall be such as to produce concrete that can be puddled readily into the corners and angles of the form and around the reinforcement without excessive spading and without undue accumulation of water or laitance on the surface. In no case shall concrete be placed which shows a slump exceding the following limits:

For Caissons	—Max. Slump 4 in.
For Heavy Walls, Slabs and Beams	— " " 7 "
For Thin Walls and Columns	— " " 9 "

The proportion of fine and coarse aggregates shall be such that the ratio of the coarse to the fine shall not be less than 1 nor more than 2, nor shall the amount of coarse material be such as to produce harshness in placing or honeycombing in the structure. When forms are removed, the surface and corners of the members shall be smooth and sound throughout.

CONTROL OF PROPORTIONS.

The methods of measuring materials shall be such that the proportion of water to cement can be closely controlled during the progress of the work and easily checked at any time by the architect or his representative.

To avoid unnecessary or haphazard changes in consistency, the aggregates shall be obtained from a source which will insure uniform quality and grading during any single day's operation, and they shall be delivered to the work and handled in such a manner that variations in moisture content will not interfere with the steady production of concrete of a reasonable degree of uniformity.

TESTS OF CONCRETE.

Frequent tests will be made by the architect throughout the work to determine the quality of concrete being produced. These tests will be made at the expense of the owner and will, in general, be made on 6 x 12-in. concrete cylinders loaded in compression at ages of 7 and 28 days, in accordance with the Standard Method of Making and Storing Specimens of Concrete in the Field (Serial Designation: C31-21) of the American Society for Testing Materials.

The contractor shall co-operate in every way to the end that concrete of the desired quality be obtained. He shall provide at cost, such housing as may be required for testing equipment and storage of test specimens; such cost to include only labor and materials actually used.

PORTLAND CEMENT.

Portland cement shall conform to the Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials (Serial Designation: C9-21).

FINE AGGREGATE.

Fine aggregate shall consist of natural sand having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances. The sand shall be of such sizes that it shall all pass a ⅜-in. sieve, at least 15 per cent shall be retained on the No. 8 sieve, and the fineness modulus shall not exceed 3.50.

COARSE AGGREGATE.

Coarse aggregate shall consist of gravel, crushed stone, crushed air-cooled blast-furnace slag weighing not less than 70 lb. per cu. ft. or other approved inert materials, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter. Coarse

aggregate shall not have more than 10 per cent finer than the No. 4 sieve and the maximum size shall not be greater than will permit proper placement.

STORAGE OF AGGREGATE.

Aggregate shall be so stored as to avoid inclusion of foreign materials. Frozen aggregate or aggregate containing lumps of frozen material shall be thawed before using.

WATER.

Water for concrete shall be from the Chicago water supply or other approved source.

MIXING CONCRETE.

The concrete shall be thoroughly mixed in a batch mixer of approved type. The mixer shall be equipped with suitable charging hopper. A water-storage and water-measuring device shall be provided. The mixing of each batch shall continue for at least one minute after all the materials are in the mixer during which time the mixer shall rotate at a peripheral speed of approximately 200 ft. per minute.

DEPOSITING CONCRETE.

Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable and in a manner that will prevent segregation of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. Concrete as it is deposited shall be puddled by means of suitable tools until forms are completely filled and reinforcement and embedded fixtures thoroughly incorporated in the mass.

Water shall be removed from excavations before the concrete is deposited, unless otherwise directed by the architect.

Concrete when deposited, shall have a temperature of not less than 40 deg. F. and not more than 120 deg. F. Concrete shall be deposited continuously and as rapidly as practicable until the unit of operation, approved by the owner's representative is completed.

In freezing weather suitable means shall be provided for maintaining the concrete at a temperature of at least 50 deg. F. for not less than 72 hours after placing. or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the architect. Salt, chemicals, or other foreign materials shall not be mixed with the concrete for the purpose of preventing freezing.

PROTECTION OF CONCRETE.

Exposed surfaces of concrete shall be protected from drying for a period of at least 7 days after being deposited.

DISCUSSION.

L. F. HARZA.—I would like to add a word of emphasis to what Mr. McMillan has said about drawing specifications in such a way that obtaining good concrete is an object on the part of the contractor. Several years ago I was engineer of a dam in Northern Wisconsin, and on the old usual form of specification there was a continual battle from the beginning to the finish of the job to get good concrete. Mr. Harza.

Recently, in connection with another dam built by the same contractors and the same superintendent under a specification similar to Mr. McMillan's, although by no means as simple and direct, I obtained the best concrete I have ever obtained with almost no argument with the contractor. The way it was done was this: a sufficient mixture was adopted, based upon preliminary tests, which would obtain the desired strength of concrete with an 8-in. slump, considering that to be the maximum slump which could be used without dangerous segregation. In the specifications it was stated that the contractor, if he would co-operate by using a smaller slump, could reduce the cement content to maintain approximately a constant water-cement ratio.

At that time I feared the effect of the moisture content of the aggregate and therefore determined a curve of the relation of slump to water-cement ratio, with the purpose of controlling the water-cement ratio through the slump as an indicator. The contractor soon found that it was an advantage to him and an economy in cost to put in relatively stiff concrete and most of the concrete went in with a slump of 4 or 5-in. and often less, with no segregation, no laitance or surface water, and tests which were made continuously showed that the concrete in every class averaged above the specified strength, and the contractor made an extra profit by making good concrete.

CHARLES E. NICHOLS.—Mr. McMillan expressed satisfaction with the specification for obtaining the desired quality of concrete. Presumably that satisfaction is from the viewpoint of the engineer who wants his strength and the degree of workability which will give a good finished appearance. The contractor has, as well as that interest, his own pocketbook in mind, and I should like to ask him whether, after his experience on this job, and putting himself in the shoes of the contractor, he would go so far as to say that in his opinion more expensive equipment which would give greater accuracy in measurement, would or would not have resulted in greater economies to the contractor? Mr. Nichols.

MR. MCMILLAN.—More expensive equipment would have resulted in a higher cement factor by controlling that fluctuation. Whether it would have resulted in a total net economy depends on the plant, the size of the Mr. McMillan.

Mr. McMillan. job and the cost of the cement. I know the contractor on the job was well satisfied with the specification.

Mr. Lovejoy. C. H. LOVEJOY.—It is sometimes necessary to overcome psychological inertia and change the specification; it varies with the building code.

Mr. McMillan. MR. MCMILLAN.—My interpretation of the building code is that 2,000-lb. concrete is desired. There is a little ambiguity there. One can use either a 1 : 2 : 4 mix or a 2,000-lb. concrete. Our results show that we got it. We believe that it complies with the liberal reading of the building code and satisfies its spirit.

Mr. Cummings. H. P. CUMMINGS.—Have you ever had any experience in applying the water-cement ratio principle to a concrete using a crushed slag aggregate? What would be the cement in the water-cement ratio if a dry porous slag was used that would absorb in itself a considerable amount of the water used?

Mr. McMillan. MR. MCMILLAN.—I have not, but I may state that I believe it could be as described by Mr. Ahlers by working out his curve. It would take some little experimental work to find out what mixes and what water-ratio you wanted on that job. Having determined that, you can work that curve through the rest of the operation. We have the question of the absorption and the aggregate in ordinary stone and gravel aggregate, and undoubtedly some of the variations shown here and in the other tests reported, have been due to the absorption of the aggregate itself. In the ordinary aggregate, that is a minor factor and one we can go into only with more refined methods.

Mr. Johnson. C. S. JOHNSON.—Would you use more refined equipment? It might be of interest to say that the contractor on the portland cement building is now investigating equipment to get more yield to the yard by this method of mixing.

Mr. McMillan. MR. MCMILLAN.—Both the contractors on this building have had previous experience with more refined equipment and presumably they knew what they were doing when they did not ship it into this job, because the job was a very small one, four or five stories.

New Experiences in Concrete Control.

By John G. Ahlers.*

An attempt to clarify the problem of concrete control by making field work much simpler and hence supervision easier. Accomplished by a contractor's organization in co-operation with engineering firms on six building operations. Using water-ratio theory only, strength is regulated by constant balance of water to cement by weight in simple device—allowances made for moisture in aggregate. An outline of field procedure and a proposed specification for concrete by this method. A discussion of the possible errors and tolerances due to variables. Conclusions reached from tests made.

Part I—Review of Water-Ratio Theory as supplied to a contractor's field operation with assumptions used.
Part 2—Consideration of Strength.
Part 3—Consideration of Economy.
Part 4—Illustrations of a job procedure.
Part 5—Proposed specification for Concrete.
Part 6—Variables caused by job conditions.
Part 7—Conclusion.

Part I

Application of water-ratio theory to field operations and assumptions based thereon.

There is accumulating a large collection of literature on the subject of design of concrete and its simple presentation or method of control. This paper is written in an attempt to clarify the problem so that in the end good uniform concrete of even strength will be obtained by simple methods and hence much less detail of supervision.

The procedure outlined in this paper was developed during the process of instructing the Barney-Ahlers field organization in control of concrete

* Barney-Ahlers Construction Corp., New York City.

by the water-cement ratio. A part of what follows is also from explanations written for engineers and architects for the purpose of gaining their co-operation when they did not have time to become familiar with or study the subject in detail.

To the following engineering firms acknowledgment is hereby given as without their co-operation and permission the valuable experiences of field operations of a theory would not have been possible.

George S. Kingsley Architect	Thomas Brothers' Warehouse Brooklyn, New York 1,845 yd. concrete.
Francisco & Jacobus Engineers and Architects	Lenox Laundry Mt. Vernon, New York 1,717 yd. concrete.
Westcott & Mapes Engineers and Industrial Architects	Patent Button Co. Waterbury, Conn. 2,474 yd. concrete.
Parker & Shaffer Engineers	National Metal Etching Co. Long Island City, N. Y. 2,170 yd. concrete.
Robert D. Kohn Architect	R. H. Macy Warehouse Long Island City, N. Y. 3,500 yd. concrete.
James J. Bevan Architect	E. Ruwe Printing Co. New York City, N. Y. 2,750 yd. concrete.

The concrete placed under their supervision was controlled by the procedure outlined below, handled during the summer and fall of 1925 by the Barney-Ahlers Construction Corporation and at the time of writing is not entirely completed.

The tests reported herewith are from part of these operations. The procedure eliminated the age old controversy between engineers and contractor as to proportions of cement to aggregate. This was because the water-ratio theory was accepted as proved and that strength is governed solely by the amount of water used per bag of cement as long as the concrete is plastic and workable and not by the ratio of cement to aggregate. The average engineer and architect do not realize, perhaps, as much as the best concrete man, how narrow are these limits of plasticity and workability, but when maintained the theory holds good in field practice.

It is well to start from a clear exposition of the facts relating to the water-ratio theory now generally accepted and the following extracts are quoted rather than referred to, as the theory must be kept in mind and used in following the procedure outlined.

From pamphlet "Design and Control of Concrete Mixtures," published by the Portland Cement Association:

WATER RATIO THEORY.

The water ratio theory is that, for given materials and conditions of manipulation, the strength of concrete is determined by the ratio of the volume of mixing water to the volume of cement (water ratio) so long as workable mixtures are obtained. In other words, if one cubic foot of water is used for each cubic foot of cement in a concrete mixture the strength at a given age is fixed regardless of what quantities of other materials are used, *so long as the mixture is plastic and workable and the aggregates are clean and made up of sound, durable particles.* The significance of this important conclusion is more readily appreciated when the cement paste is thought of as a glue binding the aggregates together. The addition of excess mixing water serves only to dilute the glue and reduce the strength. Less than 2½ gal. of water is sufficient to hydrate one sack of cement. While it is necessary to use more than this amount to produce a workable mix of concrete, any water in excess of that required to hydrate the cement reduces the strength and makes a more porous concrete.

The quantity of cement, the plastic condition or workability of the concrete (whether it is wet or dry) and the size and grading of the aggregate affect the strength of concrete only insofar as they affect the quantity of water required in the mix. The important influence of these factors has long been recognized, but the part played by each was not clearly understood until their relation to the quantity of mixing water was demonstrated.

More than 100,000 tests made at the Structural Materials Research Laboratory and numerous tests made at other laboratories have demonstrated the accuracy of the water ratio theory. For concrete made under average conditions and cured in the presence of moisture, the compressive strength may be expressed by the equation:

$$S = \frac{14000}{7^x}, \text{ in which} \qquad \text{(Equation 1)}$$

S = compressive strength of concrete at 28 days, lb. per sq. in.

x = water ratio $\left(\frac{W}{C}\right)$ (an exponent).

From the above equation it will be seen that the compressive strength (S) increases as the water ratio $\left(x = \frac{W}{C}\right)$ decreases. It should be pointed out that these constants were determined for definite conditions of test. It would not be expected that exactly the same constants would be found for other materials and other conditions of test.

In Fig. 1, curve A represents this equation and curve B, the equation

$$S = \frac{14000}{9^x} \qquad \text{(Equation 2)}$$

This latter equation differs from the first only in that the constant in the denominator is 9 instead of 7. It gives strengths approximately 500 lb. per sq. in. lower than the first, and may be considered as a reliable indication of the minimum strength to be expected where the concreting operations are not under rigid control. As an example in the use of this diagram it will be seen that if concrete is mixed with a water ratio of 1.00 (7½ gallons of water per sack of cement) the strength at 28 days which would be expected under average conditions is 2000 lb. per sq. in. (curve A) and that the minimum strength is 1500 lb. per sq. in. (curve B).

From *Concrete*, 1925.
"MORE RESEARCH WORK."
What the Bureau of Standards Is Doing to Solve Many Problems Relating to Concrete.
By Frank A. Hitchcock.

Water-Cement Ratio Fair.

The ratio of volume of water to volume of cement in a mix has been found to offer a fair index of the strength. The expression 14,000 divided by 7 raised to a power equal to the water-cement ratio, has been proposed in Bulletin 1 of the Structural Materials Research Laboratory for determining the 28-day strength and seems to be used to a considerable extent in practice for this purpose. This expression has been used as a criterion for studying the results of these tests.

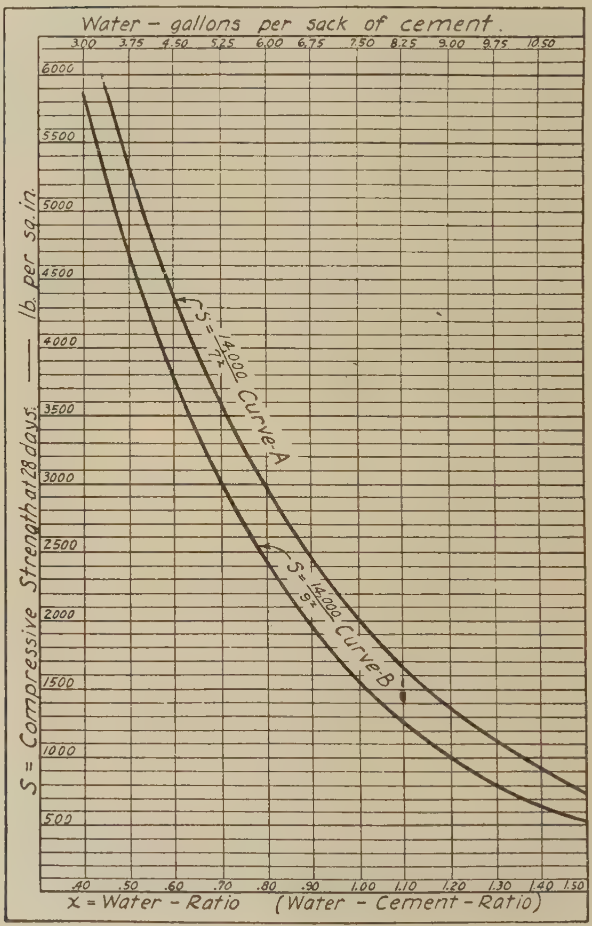


*FIG. 1.—EFFECT OF QUANTITY OF MIXING WATER ON THE STRENGTH OF CONCRETE. CURVES ARE BASED ON AVERAGE VALUES FROM NINE DIFFERENT SERIES OF TESTS MADE OVER A PERIOD OF FOUR YEARS. TESTS MADE ON ABOUT 50,000 6 X 12-IN. CYLINDERS AT 28 DAYS AT LEWIS INSTITUTE, CHICAGO.

* Copied from Bulletin Portland Cement Assn.

It was found that mixes in which the quantity of aggregate which passed through a No. 4 sieve was less than one-third of the total aggregate, as well as the mixes in which the quantity of coarse aggregate of some one size was less than one-third as great as the quantity of aggregate twice as large, generally gave strengths less than that shown by the expression used as a criterion. If the mixes referred to be eliminated, all the remaining mixes gave strengths as great as or greater than the criterion. In determining the quantity of coarse aggregate of different sizes, the aggregate was screened on a set of screens whose mesh opening advanced in size by multiples of two from the opening of a No. 4 sieve (0.094 inch) to an opening large enough to admit the largest aggregate. The amount of aggregate between any two consecutive screen sizes is referred to as of one size, and each size was considered to be twice as great as the next smaller size.

So far as these tests indicate, it may be concluded that the expression 14,000 divided by 7 raised to a power equal to the water-cement ratio, is a fair measure of the strength of concrete provided that at least one-third of the aggregate is sand (that is, smaller than a No. 4 sieve) and that the quantity of coarse aggregate of any one size is not less than one-third as great as that of the next larger size.

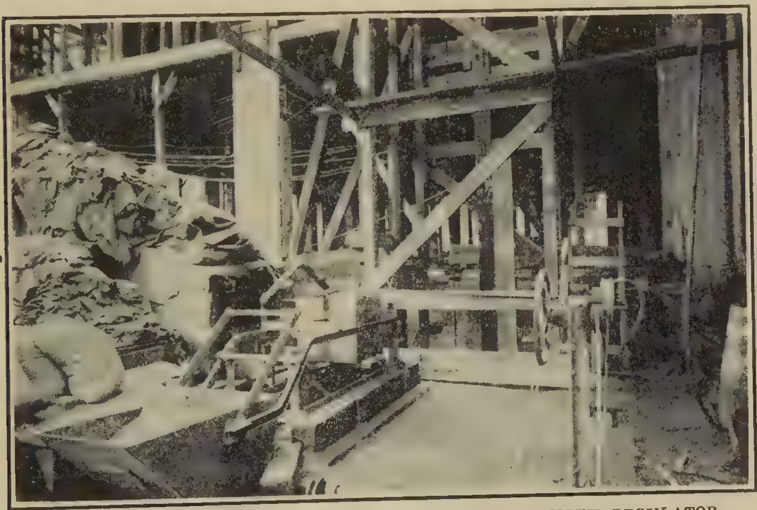


FIG. 2.—ACTUAL INSTALLATION OF CONCRETE STRENGTH REGULATOR.

Other tests included in the investigation indicated also that the sand should not be more than half the total aggregate in order to meet this criterion.

28-Day Strength from 7-Day Tests.

In the recent movement to secure strength of concrete as great at 28 days as that assumed or specified by building regulations, one of the greatest difficulties has been due to the fact that rejection of concrete which failed to meet the strength requirements was often almost impossible because of the large amount of concrete which had been placed upon it during the 28 days before it was tested. Results of recent field tests carried out on the concrete furnished in construction jobs suggested that there was a somewhat consistent relation between strengths at 7 days and those at 28 days. Further study has shown that the strength at 28 days based upon tests of laboratory specimens stored in damp sand has been equal to approximately the 7-day strength plus 30 times the square root of the 7-day strength. This relation has held true for concrete made from a large number of portland cements and for a large range of mixes. It has held true for cements whose strength was affected by variation in fineness and length of storage and for concretes with a large variety in type and quantity of admixture, such as lime, kieselguhr, tannic acid, etc. The relation

seems to be about the same for 2 x 4-in., 3 x 6-in., 6 x 12-in., and 8 x 16-in. cylinders. This relation gives promise of affording a useful criterion for estimating at an early age the 28-day compressive strength of concrete used in construction work. It is very clear, however, that the increase of tensile strength between 7 and 28 days does not follow the same law as that for compressive strength.

This application to the field testing of concrete should not be confused with the problem of determining in advance from suitable tests the adaptability, for any given work, of the concrete materials proposed for use in that work. The use of the criterion here described would not remove the necessity for these latter tests of the materials."

Having these principles firmly in mind and believing the water-ratio theory to be true, we carry out our procedure and tests, not to prove or disprove this theory, but rather we arrive at our errors and check our accuracy by comparing our results with the ideal curves as laid out by this theory which we know is fair. We start with the following:

Assumptions:

1. For workable mixes the water-cement ratio alone controls strength of concrete (using the same cement and aggregate throughout an operation). A water-cement ratio curve can be laid out for every job, therefore:
2. Measurements, calculations and tests for gradation and quantity of aggregate are not required to be more accurate than possible under ordinary field conditions. This because field errors in measurements are not nearly as great as variations caused by difference in curing or drying, and more so because the QUANTITY of aggregate used has no bearing on the strength within the limits of workability.
3. The fineness modulus is the simplest way of calculating for best yield. The combinations giving maximum density also give best gradations. It is desirable for economy and workability to use not less than ½ and not more than equal parts of fine to coarse aggregate.
4. The question of yield is a separate problem from that of strength.

Design of concrete on such a basis then becomes very simple, as it naturally divides itself into two separate and distinct phases. The first of these is STRENGTH and concerns the engineer and designer chiefly. The second is that of ECONOMY of the mixture and consequent high yield which concerns the contractor or producer of the concrete to a greater degree.

It is necessary to keep these two phases clearly apart in one's mind, as we are so apt to become confused by the old idea of regarding strength as a factor of proportion of cement to aggregate. All the facts now show that this is irrelevant and proportion of water to cement alone control strength and any grading and proportioning may be used as long as the resulting concrete is workable and plastic.

The fact that we are still in a preliminary stage of controlling concrete by this method makes it necessary for anyone not entirely at home with the subject to proceed carefully.

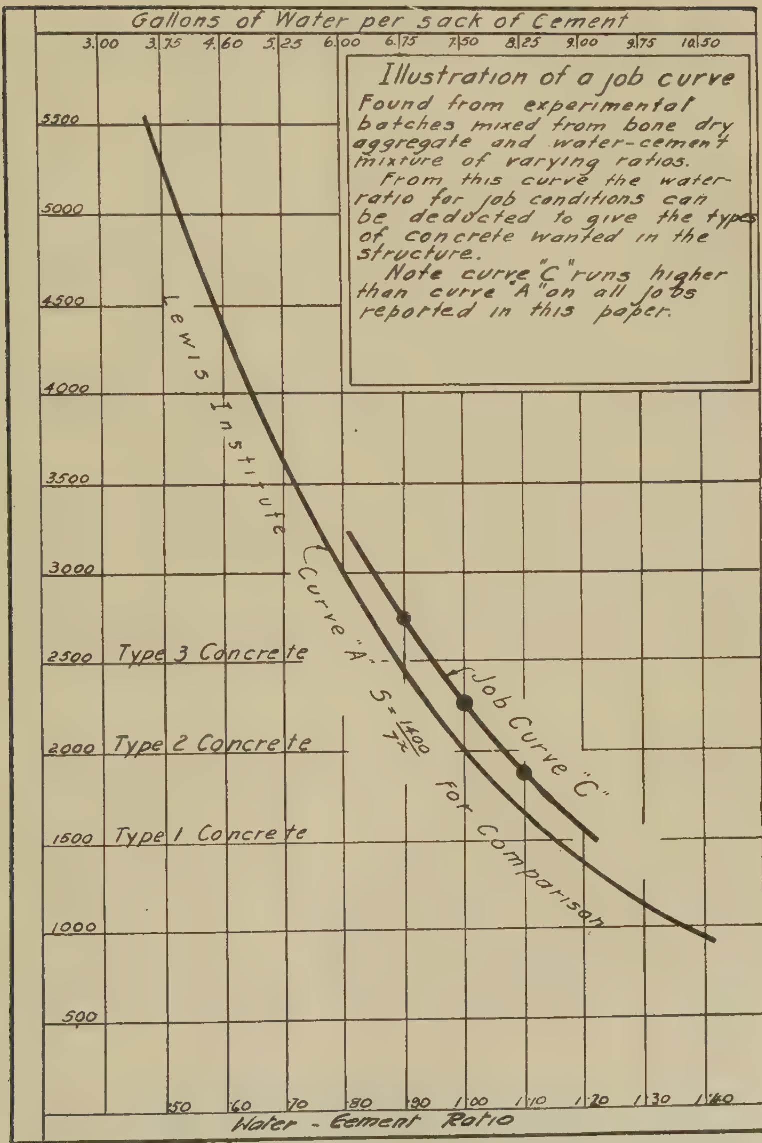


FIG. 3.—JOB CURVE DERIVED FROM EXPERIMENTAL BATCHES OF BONE-DRY AGGREGATE AND VARYING WATER-CEMENT RATIOS.

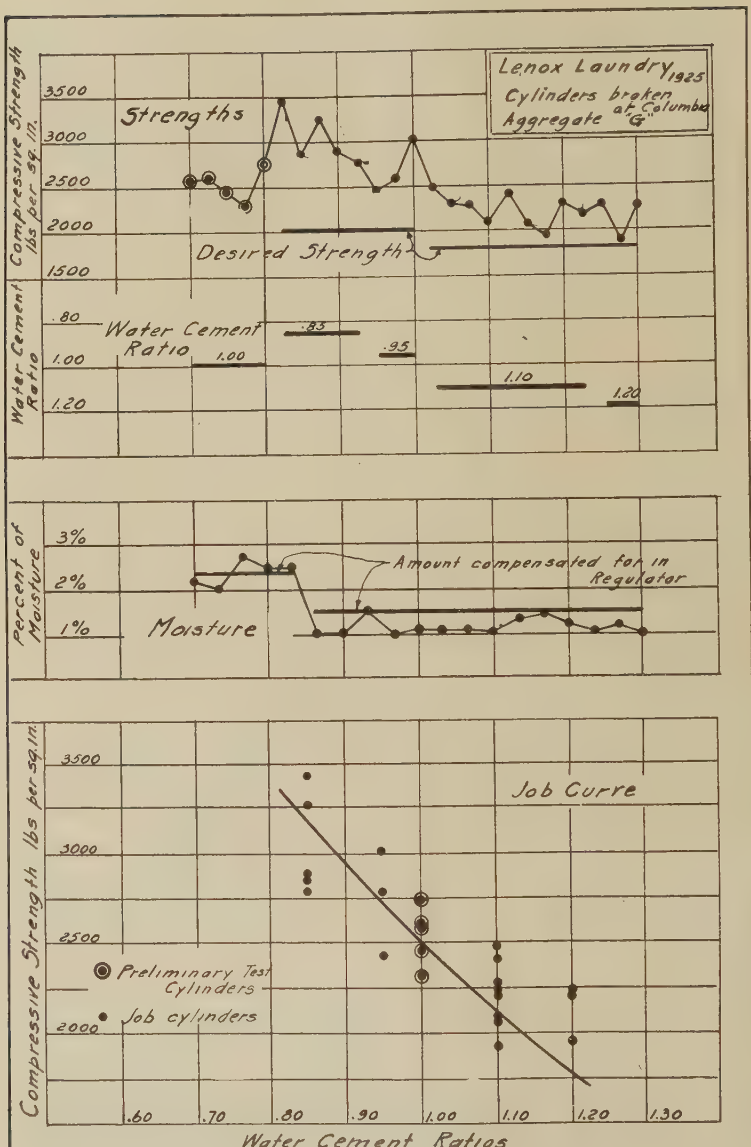


FIG. 4.—TESTS MADE ON LENOX LAUNDRY JOB.

It is not prudent to recommend elimination of measurements of aggregates or neglect control of same even though as is shown later, it is possible to produce good concrete without such control.

Let us proceed to consider these two parts separately and more in detail:.

PART II.

Consideration of Strength of Concrete.

Strength.—Strength is governed by water-cement ratio control only. If we fix this ratio for a desired strength and keep it fixed by weighing or measuring accurately separate batches of cement and water accordingly, each batch of concrete will be of the same strength when we allow for the moisture in the aggregate and use workable concrete. If the engineer can determine this ratio for his structure, he has no further work to do nor any more complications in his control of the concrete for strength. If he is sure that he is receiving aggregate similar in composition, also uniform cement, and does not allow the contractor to use concrete too dry or too wet (in other words a plastic mix) he will secure concrete uniform in strength, even though he allows the contractor to vary grading and amount of aggregate from batch to batch within the above limits of plasticity.

An engineer can determine the curve, (giving strength relation to water-ratio) for any job by mixing trial batches having such ratios as 0.9, 1.0 and 1.1. When specimens made therefrom are broken at 7 days we determine points on the job curve by using the formula S_{28} equals S_7 plus 30 $\sqrt{S_7}$ (Bureau of Standards). If he has an early start or can wait 28 days he can break his cylinders at 28 days and plot his direct results from these actual tests.

As noted in extracts quoted above, this curve will be S equals $\dfrac{1400}{A_x}$ where x equals $\dfrac{W}{C}$ *or water*-cement ratio and A may vary from job to job but will never vary within the job as long as the same quality of cement and aggregate are used.

During the early part of the job many additional samples for plotting further point on job curve should be taken, thus checking and confirming early test results, noting strength of the concrete being produced and having good control of the concrete going into the structure.

PART III.

Consideration of Economy in Concrete.

Economy-Yield.—The second phase of design, that of Economy, therefore, becomes the contractor's or producer's problem. The engineer, since his strength is fixed, can well afford to let the contractor work this out for himself, and let him combine the available aggregate in the best manner so as to obtain the greatest yield and volume from materials in his mixing plant.

Job ______ At ______ Test by ______ Week Ending ______

For this Job with Dry Aggregate Preliminary Tests

Indicate $S_7 =$ ________ $S_{28} =$ ________ for $\frac{W}{C} = 1 = \frac{7\frac{1}{2}\text{ Gals. W}}{94\text{ Lbs. C}}$

TEST SPECIMENS BROKEN

Cylinder Number	Made	$\frac{W}{C}$	Estimated Strength	Location	Mix	Fineness Modulus	Actual Strength	Equivalent to		
								7 Days	28 Days	90 Days

Moisture in Aggregate, Average ______ per cent

Daily Variations ....

Amt. Comp. for in Reg .... gallons

Average Strength, Columns

Specimens

Average Strength, Slabs

Specimens

Cylinders Made This Week

Mon.	Tues.	Wed.	Thurs.	Fri.	Sat.

Approximate .... Yards in Columns

Approximate .... Yards in Slabs

Remarks

Signed ....

FIG. 5.—FORM FOR REPORT.

This can be accomplished in two ways, either by using a dry mix, that is by increasing the volume of aggregate per batch, or by better grading.

The working condition of any job will control the dryness permissible in the concrete. The contractor, when he reached this limit, should then think about the further gains he can make from using the best graded aggregate in his mixer. He will continually be trying to add all the aggregate possible to his fixed amount of cement and water. Superintendents and foremen have found it possible to use much dryer concrete than has been the general practice in the past, when they knew they were saving thereby.

The best possible grading of aggregate can be accurately computed by the fineness modulus method. This is not a difficult way of judging aggregate. It merely involves drying some samples, weighing the amounts retained on certain sizes of screens or sieves, and adding the accumulative percentages (see Bulletin of Portland Cement Association, "Control and Design of Concrete Mixtures.")

As an example, on one job the price of sand was \$1.25 per cu. yd. and of stone \$2.40 per cu. yd. Therefore,

First.—Using a straight 1 : 2 : 4 mix the cost per cubic yard for aggregate was \$2.72.

Second.—Having determined the fineness modulus of the sand and stone and proportioning them according to the formula $P = 100 \times \frac{A\text{-}B}{A\text{-}C}$

$\text{F.M.}_c = 6.30$ $\text{F.M.}_f = 2.60$,
The proportion of sand = 39 per cent.
The cost per cubic yard now became \$2.54.

Third.—Using a coarser graded stone with the same sand and proportioning them again by the same formula:

$\text{F.M.}_c = 7.15$ $\text{F.M.}_f = 2.60$,
The proportion of sand now = 43 per cent.
The cost per cubic yard falls to \$2.48.

Apart from this saving there was also a greater yield per barrel of cement due to this better grading and therefore a lower cost per yard in cement used.

If this method seems formidable to a contractor or if he hesitates to approach the problem in a scientific way, let him dry the samples and by trial obtain the greatest weight per cubic foot by mixing and packing different proportions in a container (put in three layers and ram each layer 30 times with a rod). This heaviest combination will also be the one giving the greatest yield when mixed with water and cement, for every void is then thoroughly filled.

This may sound like common sense but it is precisely what the fineness modulus does with mathematical precision.

It is well to mention here that for practical reasons, (chiefly those of surface appearance and easy working) a little higher amount of fine material than is scientifically required will work better under field conditions.

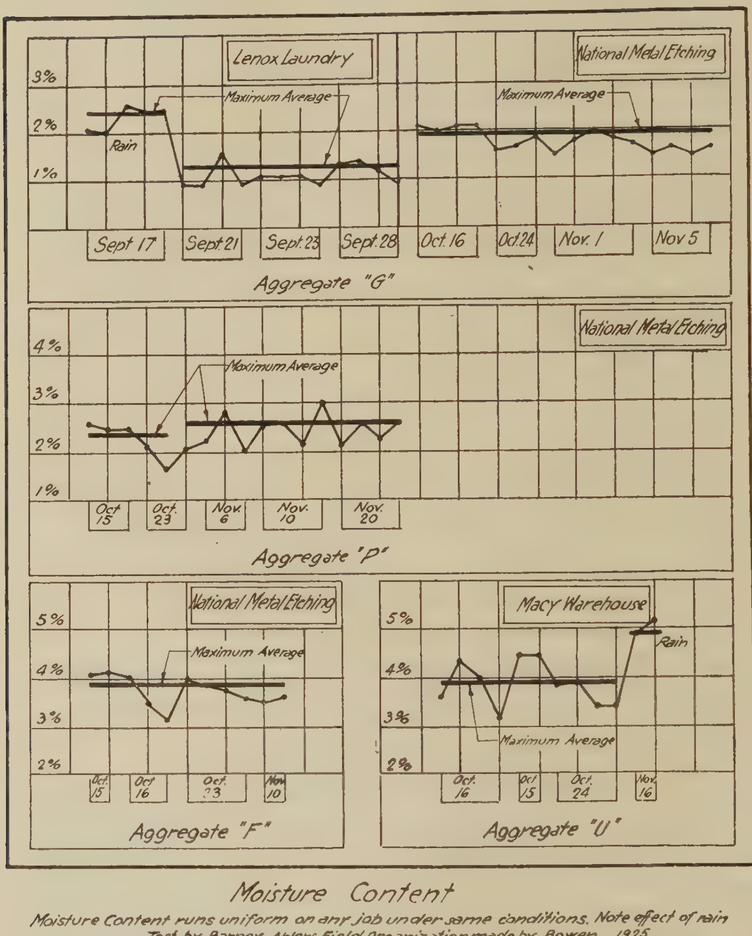


FIG. 6.—MOISTURE CONTENT ON FIVE JOBS SHOWING MAXIMUM AVERAGE.

This is in line with the desire of mechanics on a job to use more sand than is actually right for the highest yield. It is interesting to note too that it has been found very easy to make tests for the grading or combinations by using the mixer as a testing tool while it is making concrete for the job. By adding different proportions of fine and coarse to successive batches the easiest working and best appearing concrete was found. A little experi-

menting of this kind will in no way affect the strength of resulting concrete, but will actually demonstrate to the mixer operator how the water-cement ratio theory works in practice.

For instance, the use of a small amount of very fine sand containing up to 5 per cent of loam or clay sometimes actually helps the smoothness of the mix and will function the same as lime or other admixtures. In accordance with Bulletin No. 8 of the Structural Materials Research Laboratory, however, the addition of foreign admixtures is apt to decrease the strength, so the actual strength should always be checked from concrete produced with such admixtures. The addition or subtraction of fine or coarse clean sand of the same composition as balance of aggregate will not affect the strength and can be varied to suit the need of the concrete.

This concludes the consideration of Strength and Economy as separate parts and the remainder of this paper will be confined to a discussion of the practical field applications of the above theories as a whole with actual illustrations of results obtained on field operations and field tests along these lines. We will note later the possible variations in strength due to field errors, measurements, compensations for moisture, etc.

PART IV.

Illustration of a Job Procedure.

As an illustration of practical field application of above procedure let us take, say, a reinforced-concrete building operation.

We have the building about to begin, excavation almost complete, and are ready to start concreting of the foundations. The first car-load of cement has been delivered to the work and a supply of sand and gravel; or perhaps the commercially known Ready-Mix is already at hand.

Cement, Tested.—First, we make sure the cement is being and has been tested by the mill or testing laboratory so that uniform cement will be used. If possible, standard mortar compression tests on cement should be made of each shipment along with the usual A. S. T. M. tensional test.

Aggregate Samples Dried.—We next take several fairly representative samples of aggregate. First we weigh them in actual moist condition, then dry them and weigh again, noting carefully resulting loss in weight and hence obtain the average percentage of moisture. (This for later use and information.) We are now ready, say a week ahead of actual concrete operations to make tests to locate our particular job curve, that is, to find the water-cement ratios required for the grades of concrete desired.

Water-Cement Ratios.—We take and mix several small batches of water and cement together in various ratios such as:

(a) 0.9 meaning .9 cu. ft. water (6¾ gal.) per 1 cu. ft. or bag cement.
(b) 1.0 meaning 1 cu. ft. water (7½ gal.) per 1 cu. ft. or bag cement.
(c) 1.1 meaning 1.1 cu. ft. water (8.25 gal.) per 1 cu. ft. or bag cement.

Trial Batches.—To these batches of water-cement binder or grout, we add some of our dried aggregate. It should be added in about proportions such as is intended to be used on job—but the main thing is to add enough so as to get a concrete to about job conditions of plasticity. From each trial batch make compression specimens (6 x 12 or 8 x 16-in. cylinders) in accordance with A. S. T. M. specifications. Let us make at least seven of each kind. Seven days later let us break not less than four of each batch.

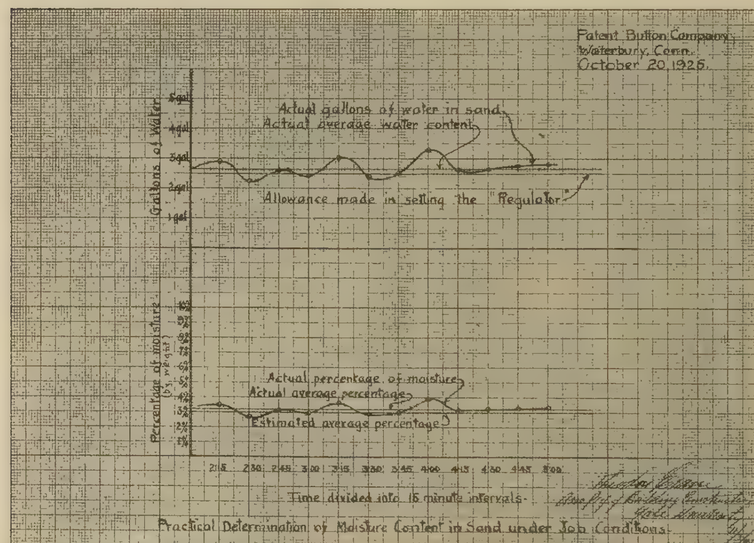


FIG. 7.—MOISTURE CONTENT OF SAND UNDER JOB CONDITIONS—PATENT BUTTON CO.

7-Day Concrete.—Let us assume we found the averages of the test cylinders as follows:

7-days—

(a) For $\frac{W}{C}$ 0.9 strength at 7 days 1540 lb. per sq. in. Average.

(b) For $\frac{W}{C}$ 1.0 strength at 7 days 1205 lb. per sq. in. Average.

(c) For $\frac{W}{C}$ 1.1 strength at 7 days 940 lb. per sq. in. Average.

28-Day Concrete.—Then from formula of Bureau of Standards as above mentioned:

$$S_{28} \text{ equals } S_7 \text{ plus } 30\sqrt{S_7}, \text{ we have}$$

$$\text{(a) For } \frac{C}{W} \quad .9 \; S_{28} \text{ equals } 1540 \text{ plus } 30\sqrt{1540}, \text{ equals } 2720 \text{ lb.}$$

$$\text{(c) For } \frac{W}{C} \; 1.0 \; S_{28} \text{ equals } 1205 \text{ plus } 30\sqrt{1205}, \text{ equals } 2245 \text{ lb.}$$

$$\text{(b) For } \frac{W}{C} \; 1.1 \; S_{28} \text{ equals } 940 \text{ plus } 30\sqrt{940}, \text{ equals } 1860 \text{ lb.}$$

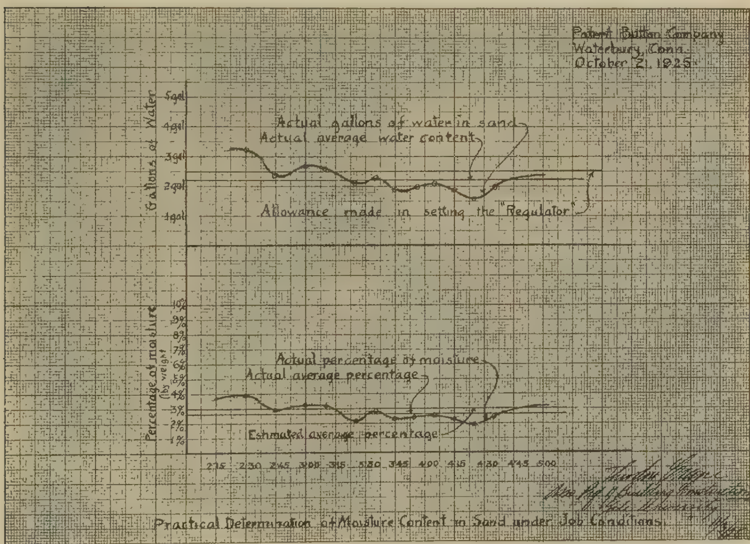


FIG. 8.—MOISTURE CONTENT OF SAND UNDER JOB CONDITIONS—PATENT BUTTON CO.

Job Curve.—This gives us three points on our job curve and we draw same as indicated on diagram, Fig. 3 and called Curve *C*.

It is interesting to note that on the tests reported herewith using quartz gravel and sand in a pre-mixed aggregate, all the curves run higher than Curve *A* in Fig. 1. (Bulletin 1 Lewis Institute.)

Desired Concrete Strengths.—We have now reached the point in our procedure where the engineer can specify definite water-cement ratios for certain parts of his work. We will assume he wishes three types of concrete:

Type 1.—Non-bearing walls, parapets, etc., ultimate strength required, 1500 lb.

Type 2.—Structural members, beams, slabs, girders, etc., ultimate strength, 2000 lb. at 28 days.

Type 3.—Columns, ultimate strength, 2500 lb. at 28 days.

Water-Cement Specification.—Looking at his plotted curve and noting the water ratios corresponding to his types of concrete his specifications will then be for:

Type 1.—1500 lb. use $\frac{W}{C} = 1.21$ or $\frac{1.21 \times 7.5 \text{ gal.}}{\text{per sack cement}} = \frac{9.075 \text{ gal.}}{\text{sack cement}}$

Type 2.—2000 lb. use $\frac{W}{C} = 1.03$ or $\frac{1.03 \times 7.5 \text{ gal.}}{\text{per sack cement}} = \frac{7.725 \text{ gal.}}{\text{sack cement}}$

Type 3.—2500 lb. use $\frac{W}{C} = 0.95$ or $\frac{0.95 \times 7.5 \text{ gal.}}{\text{per sack cement}} = \frac{7.125 \text{ gal.}}{\text{sack cement}}$

Moisture in Aggregate.—In order to use exactly this number of gallons of water in each batch it is necessary to compute the moisture content in the aggregate. Numerous tests have indicated that the average in sand runs near 3 per cent, in gravel 1½ per cent by weight.

Without going into the details of computation and errors resulting from wrong assumptions, as these are discussed later, we will say that the "*maximum average*," (see Part VI, Variable B and C) content was found for the job to be 2.5 per cent. This information was obtained in the beginning from the many dried samples. Supplementary moisture tests should be made later.

Absorption of Aggregate.—As all preliminary tests were made with bone-dry aggregate and calculations for moisture are based on same, the absorption can be ignored.

Water Control at Mixer.—We may be using Ready-Mix in a ½-yd. mixer, if so we will probably be using around 14 cu. ft. or about 1,400 lb. aggregate. The average moisture contained is then 1,400 x 2½ per cent equals 35 lb. or 4.25 gal.

If a concrete strength regulator is used and 2 bags of cement per batch, we can set same:

For Type 1 concrete, 18.150 — 4.25 gal. = 13.90 gal. per 2-bag batch.
For Type 2 concrete, 15.42 — 4.25 gal. = 11.17 gal. per 2-bag batch.
For Type 3 concrete, 14.25 — 4.25 gal. = 10.00 gal. per 2-bag batch.

The job can proceed at once and is early insured of getting uniform sound concrete. The engineer responsible for strength control at the mixer need only see that each batch contains two *full* bags of cement and the necessary computed amount of water for same.

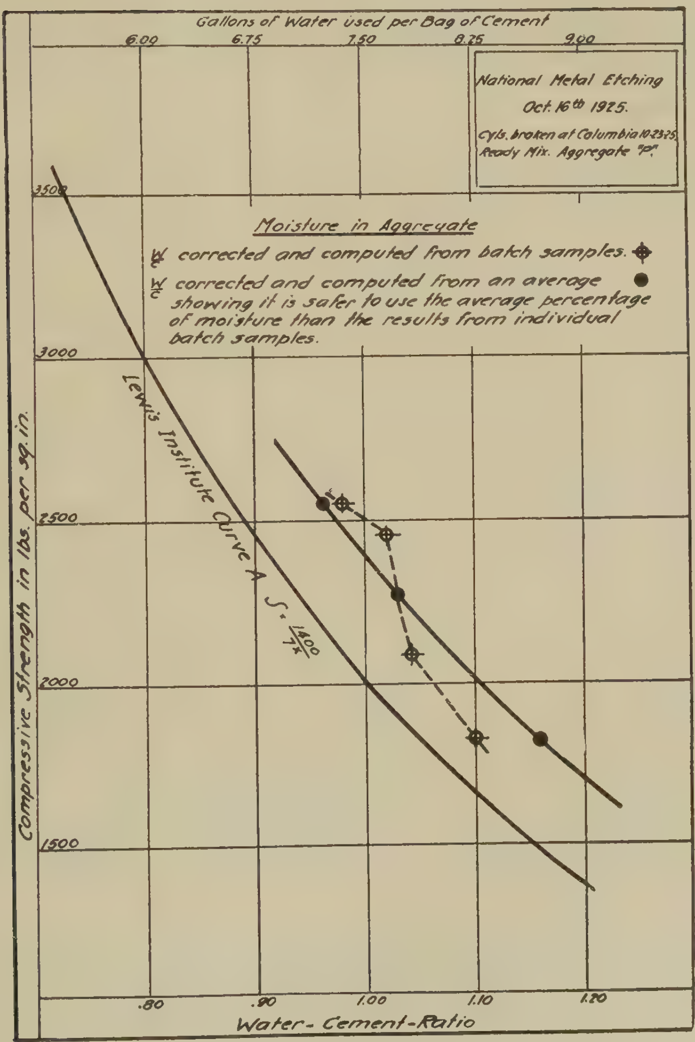


FIG. 9.—MOISTURE CONTENT OF AGGREGATE USED IN NATIONAL METAL ETCHING JOB.

Check Cylinders.—As soon as job starts and concrete is being poured cylinders must be made at least weekly to check the computations, as concrete from a mixer runs sometimes better and stronger than small batches mixed by hand and sometimes (for instance if mixed too short a time) runs weaker than test batches. If the water content in aggregate is below the assumed average the concrete will also run stronger than expected. (See Part VI, Variable C, below.)

More Points on Job Curve.—We should test these cylinders at 7 days and compute 28-day strengths. We plot these as soon as available, and we thus acquire additional points on our job curves. We also plot points from 28-day old cylinders when these are broken.

A sample of how actual results may be recorded is shown in summary of one job later. (See Fig. 4—Lenox Laundry.)

With such information constantly plotted we get an exact trend of our job as it proceeds and can keep control of the proper water ratio to use for the various types of concrete that are desired.

Variables.—The only variables that make for changes in strength are then, the moisture changes in the aggregate and the variation in cement.

Tests for Variables.—To control the cement we only have recourse to the mill tests and mortar tests on same. To control the moisture in the aggregate we may check this twice daily by drying, say, 20-lb. samples, weighing before and after drying, or by comparing with standard saturated samples.

Comparison with standard saturated samples means finding how much water is needed to fully saturate any given amount of fine aggregate. It is easy to measure or weigh how much water is needed fully to saturate any given amount of bone-dry sample of sand. Therefore, knowing the weight per cubic measure of fully saturated sand and measuring the amount added to any incompletely saturated sample fully to saturate it, the difference between what was added and what it is known to contain when fully saturated is the amount present in any job sample.

Such tests can be made in a few minutes when the operator gets used to it. Moisture contents run very uniform however, (See Fig. 6) and can easily be judged by the operator.

Yield.—To insure the maximum yield in the concrete the contractor will study his density and grading. The concrete will be of the same strength regardless of these as long as it is workable and has no voids, but the yield is maximum in volume of finished concrete when the weight per cubic foot of aggregate dry and rodded is greatest.

At the present early stage of controlling concrete by this method and to insure against abuse, it is considered desirable, where the operators are not fully experienced, to control the volume of aggregate going into each batch and to insure use of aggregates having at least 33 per cent below ¼ screen.

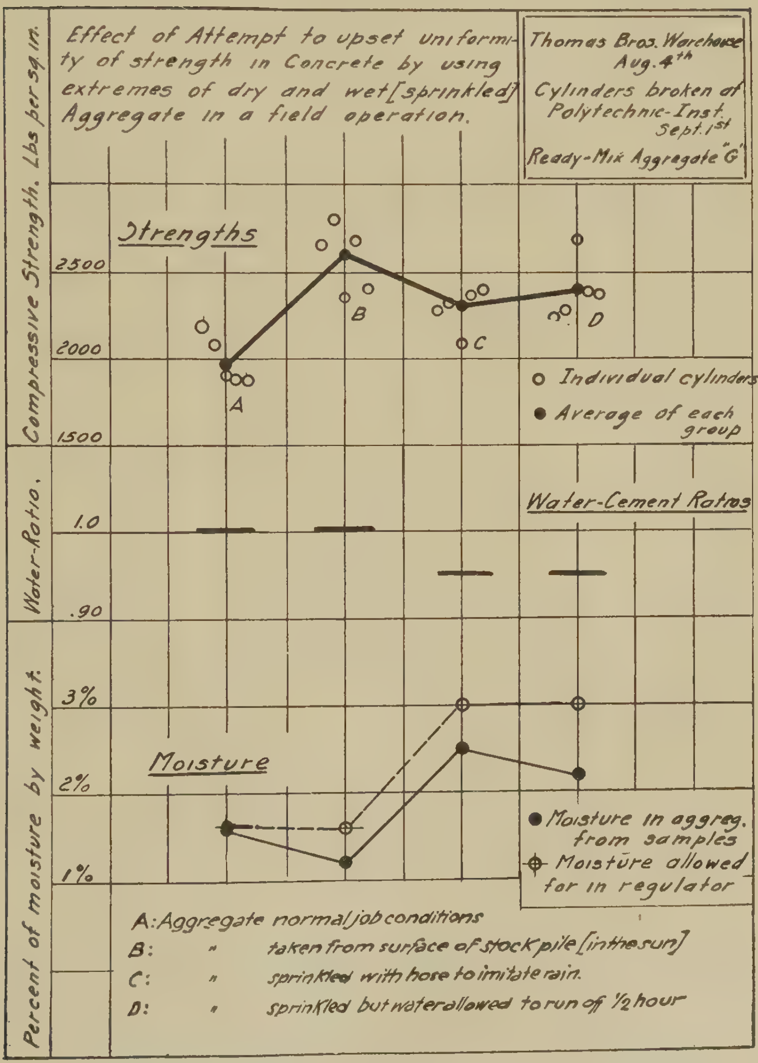


FIG. 10.—RESULTS OF TESTS MADE ON THOMAS BROS. WAREHOUSE JOB.

Gradings for Economy.—It is essential that the fine and coarse aggregate be well graded within themselves as well as in combination. It is possible to improve a sand by better grading or screening. It is possible to improve gravel or broken stone aggregate by grading from coarse to say ¼ in. The resulting combination of fine and coarse is then much improved and can be proportioned for maximum yield.

This can be done by making a sieve analysis as indicated in the Portland Cement Association Bulletin or Lewis Institute Bulletin No. 1.

To illustrate by an actual case at the Patent Button Job. Using, at first, broken stone and sand in a straight 1: 2: 4 mix the yield was 0.74 yd. of concrete per barrel of cement with an almost unworkable concrete due to all coarse stone being furnished. The stone was then graded down to ¼ in. The fineness modulus of the sand and stone was determined and the proportions of each to a batch computed: F.M. = 7.23, F.M. = 3.75. The proportion of sand computed was 45 per cent; the yield, 0.79 yd. of concrete per barrel cement—a saving of 5 per cent in the cost per yard of concrete, apart from a further saving in getting more concrete out of a yard of aggregate.

PART V.

Proposed Specifications for Concrete.

If it is desired to write a specification for producing concrete under this procedure the following is suggested:

The engineer writing such a specification will have full control of the strength of the concrete going into the work, and will hold forth a promise of saving to the contractor by increased yield and saving in cost when following the specification.

PROPOSED SPECIFICATION.

Cement. Cement shall be an approved brand of Portland Cement and shall be tested in accordance with A. S. T. M. Standards for tension and compression. Must show a strength in compression of not less than 2800 lb. per sq. in. in 28 days.

Aggregates. *Aggregates* to be composed of clean, hard, strong, durable, uncoated particles free from injurious amounts of deleterious matter. Shall consist of fine and coarse or a combination of same such as will give a fineness modulus in combination varying not more than 10 per cent above or 15 per cent below the values given in Table III, Bulletin 1, of The Structural Materials Research Laboratory, "Design of Concrete Mixtures."

The proportion of fine to coarse will be left to the Contractor as he complies with the other parts of this specification and produces a controlled workable mixture of concrete.

Preliminary Tests. The Contractor will under direction and supervision of the engineer, make preliminary tests before starting actual concrete operation.

If these preliminary tests are not made in time the proportions of cement to water and to aggregate must be used as required by the Engineer.

Preliminary Test Cylinders. Preliminary Test Cylinders shall be made from bone-dry aggregate in small batches of definite ratios of water to cement (not less than 3 ratios and 7 cylinders of each).

These cylinders to be broken at 7 and 28 days and will govern until further test specimens are made. The water-cement ratio required to produce the strengths desired shall be computed from these tests.

Strengths. The following strengths are desired:

Type 1.—Non-bearing walls, curtain walls, slabs on fill, etc.—1500 lb. at 28 days.

Type 2.—Structural members, beams, girders, slabs, etc.—2000 lb. at 28 days.

Type 3.—Columns, 2500 lb. at 28 days.

The volume of water per bag of cement arrived at must be controlled by some approved device regulating the water-cement ratio, with due allowance for moisture in the aggregate.

Supplementary Tests. Early in the job additional concrete cylinders shall be made (not less than 7 for each 200 cu. yd. of concrete) until 1000 yd. have been poured.

Then not less than 7 cylinders must be made for every 500 yd. These cylinders to be cured and broken under A. S. T. M. Standard Specifications and under direction of the Engineer.

Changes in Water Cement Ratio. The Engineer shall have the power to change the ratio of water per bag of cement to conform with results of supplementary tests.

Moisture in Aggregate. If desired by the Engineer the Contractor shall make two tests daily, to determine the average moisture content of the aggregate.

Proportions. When so controlled the Contractor will be permitted to use his judgment as to proportions of fine to coarse aggregate, and amount of aggregate, providing however, 33 per cent of aggregate is below ¼-in. screen.

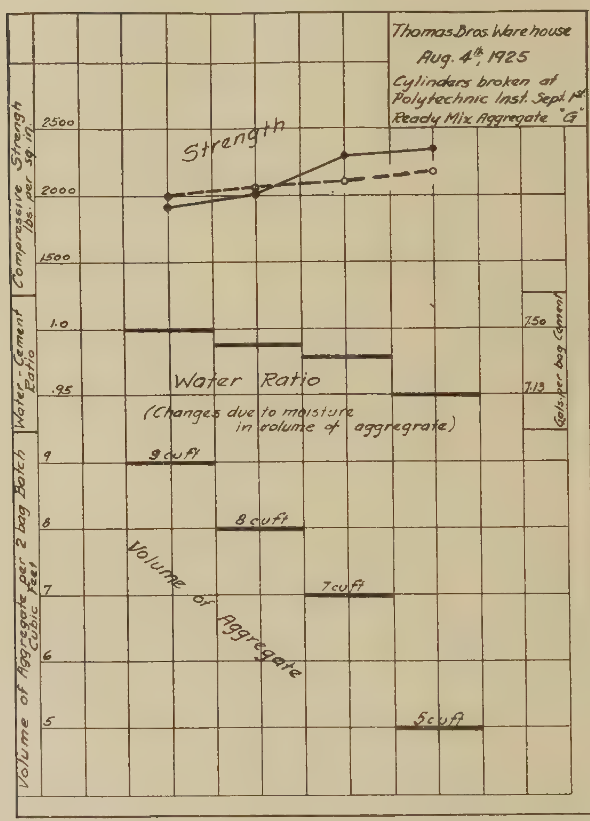
The slump must be great enough to allow the concrete to give smooth surfaces when tamped against forms and uniform contact with reinforcement when properly rammed in place (not less than 4 in. nor more than 8 in.).

Engineer's Report. Full data on all tests and cylinders made and broken to be furnished the Engineer in charge.

PART VI.

Variables Caused by Field Conditions.

Errors and Variations.—When controlling concrete by using the water-cement ratio theory, certain errors in operation and tests may occur, and daily and hourly variables must be considered.



Practical Field Demonstration of Water-Ratio Theory
By Professor W. J. Moore, Polytechnic Inst.

Comparative small decrease in strength (13 %) against large increase in volume of aggregate (80%) keeping Concrete-Strength-Regulator set for fixed amount of water
Three batches made of each proportion and five cylinders made from the third batch
From Lewis Institute, Curve "C" would expect an decrease in Strength as shown by dotted line (Due to total moisture reduction)

FIG. 11.—FIELD DEMONSTRATION OF WATER-RATIO THEORY.

The important variables to study are the following:

(a) Errors resulting from incorrectly balanced amounts of water and cement getting into successive batches—variation in water-cement ratio.
(b) Variation in moisture content in aggregate.
(c) Errors in assuming an average moisture content.
(d) Errors in moisture due to change in amount of aggregate used.
(e) Variations in test cylinders.
(f) Variations in cement.
(g) Change in chemical composition or intrusion of impurities in aggregate.
(h) Change in grading of aggregate.

Each of these will be considered separately and weighed in the light of actual experiences in the field, and some test data will be shown to confirm the conclusions reached later.

The tiresome long tables of test results have been omitted, but it is perhaps well to mention that back of all the curves and data presented stand the test specimens made on the jobs and broken at the laboratories of Columbia, Yale and Polytechnic Institute. Much of the test work has been done in collaboration with Professors Krefeld, Crane and Moore from these institutions.

A.—*Variation in Water-Cement Ratio—From Batch to Batch.*—This variation can be considerable as inspection of plants not controlled by special devices has indicated. Note for instance, that a cement bag improperly shaken will contain cement varying from 1 to 3 lb., therefore, in a two-bag batch there may be as much as 5 lb. variation from batch to batch. Also, there are often broken bags.

The best water measuring tanks may vary as much as 2 or 3 qt. per batch.

Let us see how much effect these variations have on the strengths of concrete by changing the water ratio. Say there is a bag slightly torn or spilled or improperly shaken, assume 9 lb., one-tenth of one bag lost. For $\frac{W}{C}$ equals 1 in a 2-bag mixer, the water ratio would then be increased to $\frac{\text{2 cu. ft. water}}{\text{1.9 bags cement}}$ or, $\frac{W}{C}$ would be 1.05 from Curve C, Fig. 3. This would indicate a decrease of strength of 200 lb. for the batch.

Even if the bags were only improperly shaken, there might be 5 lb. or one-twentieth of a bag left out, a decrease in strength as above calculated of 100 lb.

Let us note the variation caused by an improper measuring device that might be 3 qt. over, a not uncommon occurrence.

This will give a variation as follows:

$$\text{Suppose } \frac{W}{C} = 1 \text{ or } \frac{7\frac{1}{2}\text{ gal. water}}{\text{1 cu. ft. cement}} \text{ or } \frac{\text{15 gal. water}}{\text{2 bags cement}}$$

then when 3 qt. over,

$\frac{W}{C}$ will become $\frac{15\frac{3}{4}\text{ gal. water}}{2\text{ cu. ft. cement}}$; $\frac{15\frac{3}{4}\text{ gal.}}{2\text{ bags}}$ or $\frac{1.05\text{ cu. ft. water}}{1\text{ cu. ft. cement}}$ or 1.05.

From Curve *C*, Fig. 3, this would give a decrease of 200 lb. per sq. in. in strength of resulting concrete.

With such definite effects in strength from merely improper shaking of bags and inaccuracy in water, is it any wonder that under the good old ways we had erratic results and needed tremendous factors of safety?

Here is a possible decrease in strength:

From inaccuracy in weighing water............	200 lb.
From improperly shaking bags.................	100 lb.
	300 lb.

or 15 per cent in a desired 2,000-lb. concrete.

To overcome this variable the writer has developed a simple device that while a proprietary article is so illustrative of the new way of making concrete and so simple of explanation and necessary for sure control that he has taken the liberty to write a description of this so-called "Concrete Strength Regulator" as Appendix A to this paper.

The device merely balances each batch of cement in a hopper against the desired amount of water, thus:

By weight $\frac{W}{C} = \frac{a}{b}$ the water-cement ratio.

The water shuts off when in balance, therefore, as long as lever arms *a* and *b* are unchanged, the ratio of water to cement must remain constant no matter if broken bags are used or bags improperly shaken or cement spilled. All the tests reported herewith were made on jobs where this device was used so any errors or variations in the water-cement ratio itself have definitely been overcome and need not be considered in these tests. This is considered a contributing cause to the satisfactory results in producing concrete under these new methods.

B.—*Moisture Variation.*—The change from day to day in moisture content is small and less than commonly assumed.

The accompanying charts indicate actual test for moisture over a long period and all sorts of weather conditions. (See Fig. 6.)

There is also attached, charts of special tests for moisture in sand by Professor Crane, Yale University. These give water content in sand both in percentages and in gallons per batch. (Figs. 7 and 8.)

From all the observations made, it has become the experience of the

organization making these tests that a so-called maximum average could be arrived at that would be safe.

There is a characteristic maximum average for each aggregate that holds good from job to job and the deviation from this is small and can be ignored, and as it is taken above the actual average, the greater amount of batches will usually be drier and hence stronger than expected.

The variation is so small, in fact, that in a series of tests when the apparent variations (as shown from dried samples) was corrected for in water-cement strength curve the curve became broken and irregular. When the percentage of moisture was computed from an average amount for all the batches, the water-cement strength curve immediately became a smooth line. (See Fig. 9.)

C.—*Probable Errors from Assuming Average Moisture Content.*—The *probable errors* from assuming a *maximum average moisture* content can be computed.

As stated above, all batches of concrete where moisture is less than maximum average will run stronger than computed and can, therefore, be forgotten. Actual job tests, such as a series by Professor Moore of Polytechnic Institute, show this. (See Fig. 10.)

We note here the effect of selecting some dry aggregate and later sprinkling same aggregate and correcting for an assumed rainy condition, and all the resulting concrete ran stronger than required.

All tests indicate that (as in Fig. 9) the moisture content is uniform and only varies gradually.

Allowing, however, that there might occur occasional moist batches of aggregate, we will attempt to compute decrease in strength due to such a moist batch based on tests of aggregates for moisture.

In Figs. 10 and 11 the three worst samples contain ¾ gal. water in excess of amount allowed for.

A change in $\frac{W}{C} = 1, \frac{15\text{ gal.}}{2\text{ bags}}$ to $\frac{15\frac{3}{4}\text{ gal.}}{2\text{ bags}} = \frac{2.1}{2} = 1.05$, or from Curve *C*. Fig. 3, a loss in strength of 200 lb.

Fig. 11 shows moisture tests over long periods where only occasional tests show ½ per cent moisture over average and such occasional averages on small quantities of aggregate are likely in excess of the average content of entire batch as Fig. 11 bears out.

Such an increase of ½ per cent in moisture would change water-ratio on 1,400 lb. of aggregate @ ½ per cent = 7 lb. water from $\frac{W}{C} = 1$ $\frac{2\text{ cu. ft.}}{2\text{ bags}}$ to $\frac{15\text{ gal.} + \frac{7}{8}}{2\text{ bags}} = \frac{2.12}{2}$ or 1.06, an expected decrease in strength of 200 lb. The actual tests on cylinders have, however, not borne out that there was any such violent change in moisture of aggregate.

In all but very rainy weather the maximum decrease in strength will

be that due to an increase in water-cement ratio from 1.00 to 1.01, making 40 lb. (2 per cent), much less than the mean variation in strength of concrete cylinders on such carefully made tests as reported by New York contractors and The Joint Committee. (See *Proceedings*, American Concrete Institute, 1924, "Field Tests on Concrete," which gave a deviation of 8.2 per cent for best possible field condition at 28 days. Table II, p. 27.)

It will readily be seen that it is satisfactory to use the maximum average, for whenever dry batches of aggregate are used, the strength will always be greater than that required and then when the resulting specimens give strength averaging in excess of requirements, the water-ratio will be increased and new results in strength obtained that will be nearer the minimum set for the job. This error will, therefore, eradicate itself.

The chances are, however, as the above mentioned tests seemed to indicate, that the higher variations in moisture from average were errors due more to smallness of samples than variations in total moisture contained in bulk of aggregate that went into the mixer.

It should be made a practice, however, to compute the water content in the aggregate from time to time during the day and make adjustments by hanging small weights on the water container. The operator of the mixing plant can become so used to judging the water content of the aggregate that the possibility of error from this source may be overcome by him on inspection.

D.—*Variation in Volume of Mixture.*—About 14,000 yd. of concrete were produced under the system of control outlined above. No minute measurements of aggregate were attempted. Rather, the water-cement ratio being fixed, any change in desired consistency was made by varying amount of aggregate. The results were always satisfactory. (See Fig. 4, Lenox Laundry.)

One cubic foot more or less has no appreciable effect in water content (only 2 to 2½ lb.). The change in consistency and workability is noticeable, however, at once and as concrete is controlled entirely by its workability, this forbids excessive amounts of aggregate being added.

If as much as 4 cu. ft. are left out of a batch, (say, for instance, to obtain a wet mixture for thin walls) the effect is noticeable. (See Fig. 11). Note in this series of tests the slight change in strength as compared with the large change in volume. The change in strength due to this variable decrease of 4 cu. ft. to a batch: average moisture 2.5 per cent by weight: weight of aggregate 110 lb. per cu. ft. loose wet.

$440 \times 2\frac{1}{2}$ per cent $= 11$ lb. water $= 1.33$ gal. (or 0.117 cu. ft) of water contained in aggregate.

Where the desired water-cement ratio was unity, $\frac{W}{C}$ will now become, for a two-bag batch, $\frac{2 - 0.177}{2}$, 0.91 causing an apparent increase in strength of 400 lb. per sq. in. This checks with variation shown in Fig. 11.

E.—*Variations in Test Cylinders.*—A great deal has been and will be said on the subject of variations in test cylinders. There is a large variation from cylinder to cylinder within the same batch. It is the opinion of the writer that the higher strength cylinders are more indicative of the concrete in the structure than the low. This conclusion is reached after trying to trace the cause for low breaks and from study of published data on core borings as compared with molded specimens.

As an illustration, in Fig. 12 some of the data from H. S. Mattimore's paper in *Engineering News-Record*, Jan. 12, 1922, on "Relation Between Molded and Core Concrete Specimens" has been analyzed for comparisons of maximum, mean and minimum values. The 28-day strengths given in this paper were used as a basis for computing what the probable strength

	Molded Specimens at 28 days	Computed Strength at 300 days	Core Borings at 300 days	Series No.
Minimum	2211	3320	3980	
Maximum	4054	6150	6070	2
Mean	3095	4680	4545	
Minimum	2115	3200	3860	
Maximum	3315	5000	5452	3
Mean	2788	4220	4706	
Minimum	2229	3370	4185	
Maximum	4357	6590	6585	4
Mean	3368	5100	5288	

FIG. 12.—COMPARISON OF CORE BORINGS AND MOLDED SPECIMENS.

Taken from article by H. S. Mattimore, *Engineering News Record*, Jan. 12, 1922.

of these would be at 300 days from the parabolic curve of strength increase. These expected strengths are shown in the second column.

The actual strengths from cores as reported in Mr. Mattimore's paper are in the third column. As will be seen, the maximum and mean run very close, as close as may reasonably be expected, but the minimum in molded specimens is much below minimum in cores. This would indicate that minimums in molded specimens are accidents due to causes not present in concrete in the field concrete.

These accidents, it is the writer's belief, are caused by either improper procedure in making, in curing, in capping, or are due to large pieces of aggregate being near the surface of comparatively small cylinders.

It is recommended, therefore, that all specimens running more than 10 per cent below the average of those made from the same batch be rejected and a new average be computed. A. S. T. M. procedure allows rejection of specimens varying more than 15 per cent.

It is for this reason too, that the practice of this organization is to

make not less than five specimens at any one time and preferably 7. (This allows 4 breaks at 7 days, 3 at 28 days.)

It is recommended also that 8 x 16-in. cylinders be used in preference to 6 x 12-in. cylinders. (Bulletin No. 8, Lewis Institute.)

The proposed equation of Mr. Slater $S_{28} = 30 \sqrt{S_7}$ has been found to hold very good and has been used to great advantage in all these tests.

To have permanent records of the conditions under which cylinders are made it is important to have complete yield data. Therefore, some such uniform report as shown in Fig. 5 is desirable in order that the supervising engineer and the contractor may have a comprehensive and clear summary of the strength of the concrete in the structure.

The necessity for proper curing goes without saying. The specimens must be kept moist at 70 deg. F. to insure uniform setting and fair comparison. This must specially be watched in the cool months of the year.

F.—*Variation in Cement.*—Cement will vary in compressive strength from shipment to shipment and even from bag to bag within one carload.

This variable is well watched and controlled by the cement manufacturer. The variation caused thereby cannot be overcome by the supervising engineer or the contractor. It can be allowed for, however, when known and record should be made of this variable.

The knowledge that such a variable exists is the reason for insisting on standard compression tests on the cement in the proposed specification. The variation in compression strength of cement does not follow the changes in tension briquettes under standard tests now used.

Portland Cement Association Standards should include a compression test which is easily made and is not expensive, for concrete will be produced more and more in the future on a strength basis.

The necessity for consideration of this variable was brought home to this organization when some apparent irregularities occurred during a major test on the National Metal Etching job Oct. 16. All cylinders were made from the same carload of cement using the same water-cement ratio and a strength regulator. As the suggestion of Prof. Krefeld of Columbia, who supervised this test, samples of cement had been taken of each batch.

Later, when the cylinders were broken and cement compression tests were available the variations of some of the groups could be explained from the cement tests.

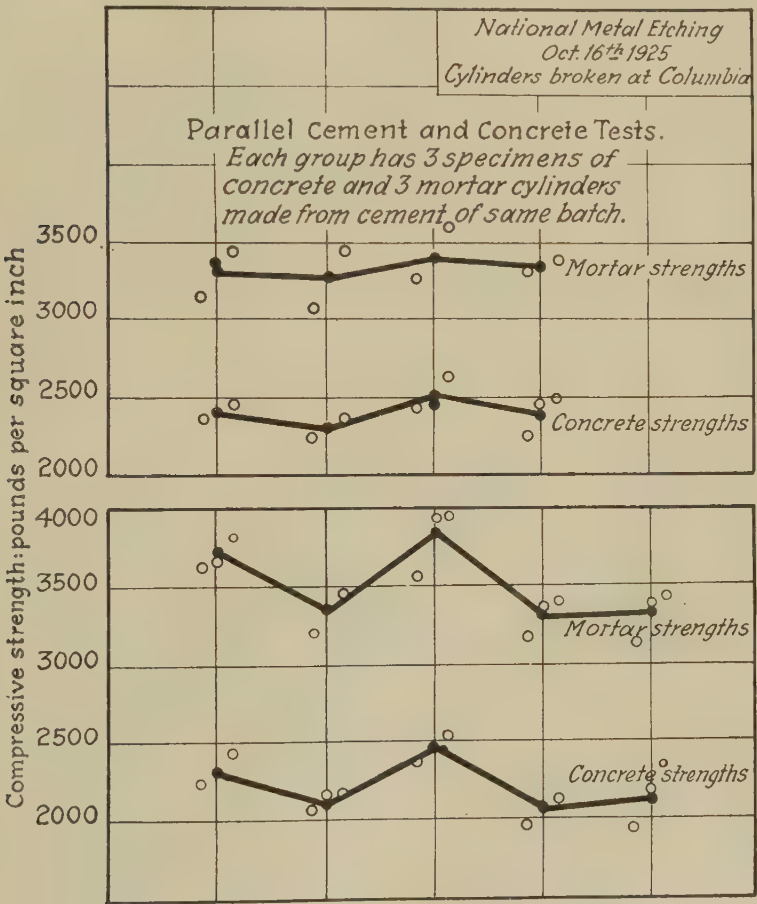
It is only fair to say that on many other groups this parallel variation did not hold good (likely because other variables intruded) and the batches shown in Figs. 13 and 14 were selected to confirm the point because of this parallel variation.

It is further shown in the tests conducted by this organization and many other tests that the difference in strength at early ages tends to narrow down at later dates.

An engineer should have this variable in mind, however, and check his

variation in concrete cylinders with possible variations in cement compression tests.

G.—*Change in Composition of Aggregate, Intrusion of Foreign Substances*.—The tests conducted by this organization indicated that aggre-



FIGS. 13 AND 14.—EFFECT OF VARIATION IN CEMENT.

Concrete specimens made under identical job conditions as to water-ratio and time of making.

All cement from same car and sample taken from each batch and tested concurrently.

gates from one definite source of supply run very uniform to the curve along with changes in water-cement ratio.

This organization in fact has acquired familiarity with several aggregates as shown in Fig. 15. It is possible to start with this information on

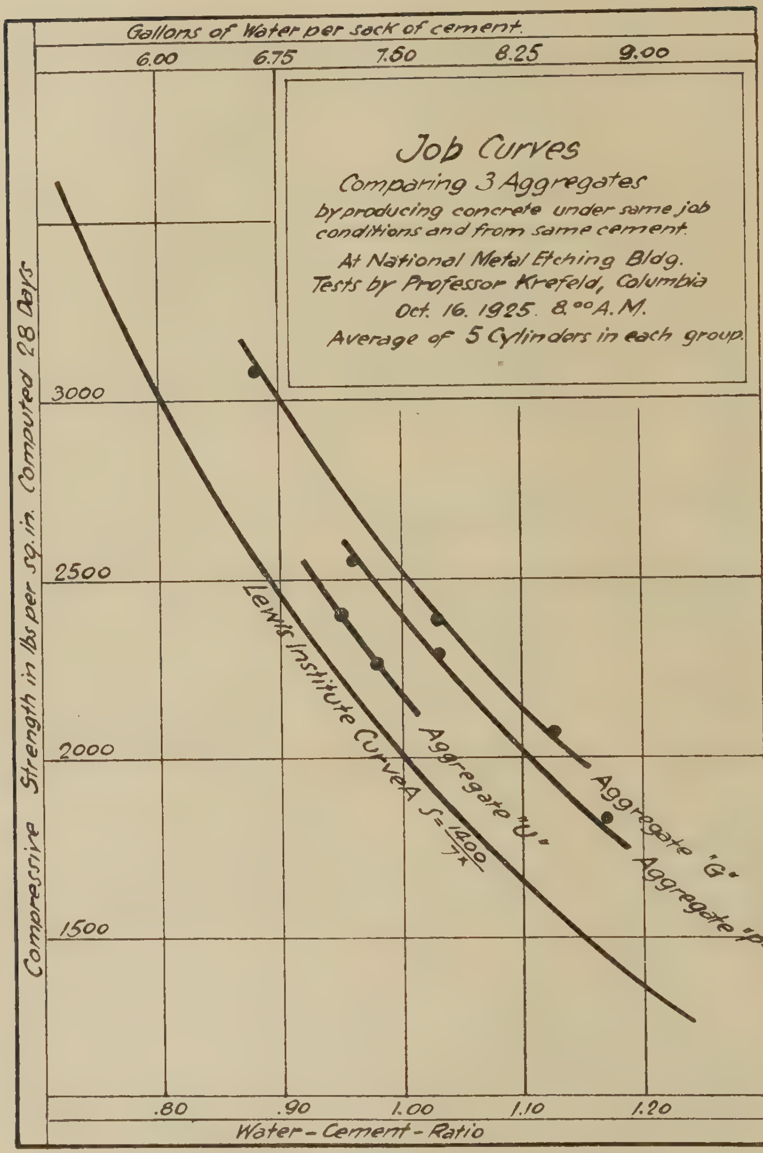


FIG. 15.—JOB CURVE COMPARING THREE AGGREGATES.

any new job and select aggregates on a value basis and judge them in view of this as to their relative costs.

In Fig. 15 all tests were made under identical conditions of plant and cement the water-cement ratio alone being varied. This is another demonstration of the value of field use of the water-cement ratio theory. A fourth aggregate tested runs so close as to coincide with one of the curves and was omitted from the chart to avoid confusion.

This change in strength curves due to aggregate composition or characteristic has been referred to in other parts of this paper. By change in composition is meant a change from broken stone to gravel or from one sand bank to another. As long as the same source is used throughout a job there would be no change in strength caused by the aggregate itself. When the type or kind of aggregate is changed a new job curve may be expected and tests must be made to see if this exists.

Regarding intrusion of impurities that is a problem to combat with physical inspection. Such intrusions are really in the nature of accidents and may occur under any conditions. If sufficient control specimens (as recommended in proposed specifications) are made of the concrete and these broken at seven days, there will be definite danger signals. When these are heeded and traced to their source, expense and trouble can be avoided.

H.—*Change in Grading of Aggregate.*—The water-ratio theory quoted in the beginning of this paper teaches that grading has no effect on the strength of the concrete as long as the resulting concrete is plastic and workable. This point has been the hardest to prove. Old customs resist new ideas and traditions and prejudices are not always broken down by facts and figures. The truth of the theory will prevail in time, however, like all new and better things even though slowly at first.

On all jobs mentioned in this paper there was no attempt made to absolutely control the grading of the aggregate. On four of these jobs Ready-Mix was used, the plant producing same separated the fine and coarse and recombined in fixed ratios. Where separate aggregates were used the stone was graded as much as possible from ¼ in. to coarse. As large gravel or stone was used as possible to increase the yield and to use higher fineness moduli. The limits were the possibility of working into place and spacing of reinforcing bars. There was also resistance from custom on this point.

Change in grading was noticed to effect the plasticity at times but rarely enough to effect the workability. There was a definite demonstration that ready mix makes just as good concrete as separate sand and gravel—sometimes better and always cheaper. Resulting concrete specimens proved that when the water-ratio theory is followed the change in grading from batch to batch has no effect on the strength of the concrete as long as it was workable and plastic. The amount of aggregate added was changed at times to keep within the limits of plasticity but the result-

ing variation in strength was caused by the contained moisture change as discussed above.

The desirability of using sufficient sand was watched so as to keep the fineness modulus as near the maximum allowed by tables found in Bulletin No. 1, Lewis Institute. Where lack of fine was apparent or lack of any size of gravel, attempts were made to supply these, either by adding at the bins or directly into the mixer.

PART VII.

Conclusions.

The water-cement ratio theory has been given a fair trial under job conditions by a contractor's organization and under the eyes of many engi-

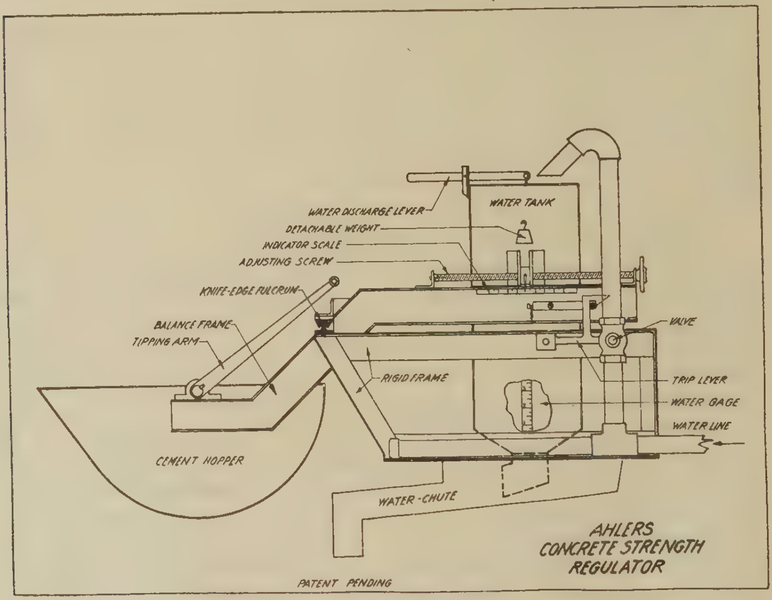


FIG. 16.—SECTIONAL VIEW OF REGULATOR.

neers. Attempt was made to work for progress and knowledge without prejudice for or against any certain materials, tools or method of manipulation. As a result the writer firmly believes that we are approaching much better and simpler means of field control of concrete and in closing would like to emphasize the following points:

1—The water-ratio theory of concrete control is practical and can be applied to field conditions for producing concrete of uniform strength and predetermined strength within reasonable limits.

2—The method of field procedure outlined herewith affords such a simple and economical method of controlling the quality of concrete, that where understood, engineers, architects and contractors have welcomed it.

3—On six building operations the new methods in this paper produced

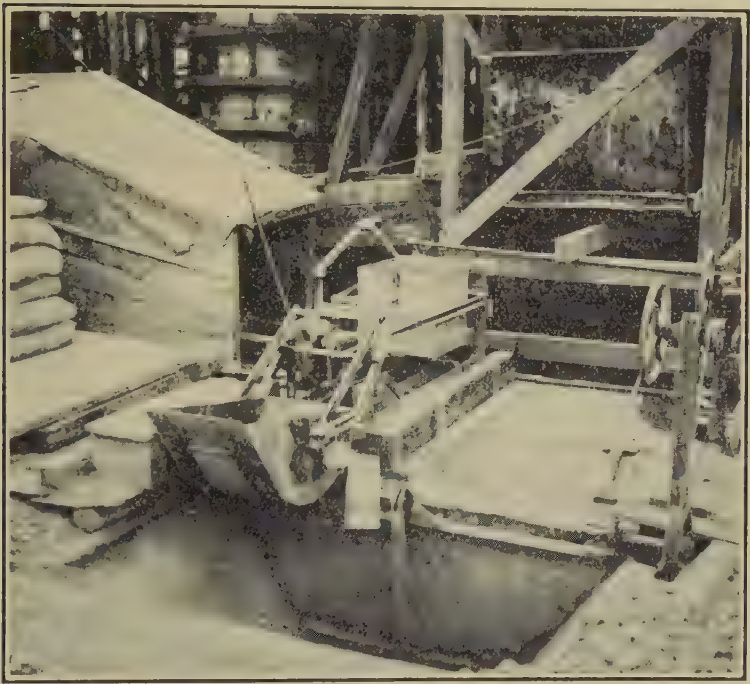


FIG. 18.—REGULATOR IN USE ON JOB.

decidedly more uniform results than shown by tests from former field conditions reported by Ahlers and Walker, American Concrete Institute *Proceedings*, 1924.

Appendix A.

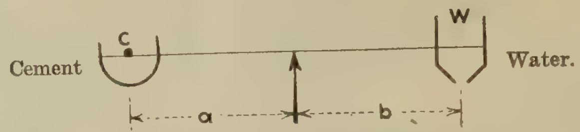
The device or tool used to control the water-cement ratio in the foregoing article has been developed and improved during the past year and has reached the stage of a practical piece of contractors' plant.

This machine is called a Concrete Strength Regulator, as that is its function and purpose.

> The water-ratio theory: . . . that, for given materials and conditions of manipulation, the strength of concrete is determined by the ratio of the volume of mixing water to the volume of cement (water ratio) so long as workable mixtures are obtained. . . .*

explains in itself the necessity for such an appliance, taking into consideration the fact that variations in the amount of cement, for a given ratio, must be compensated for AUTOMATICALLY.

It is in principle a balance, the weight or volume of water being balanced against that of cement.



By weight $\frac{W}{C} = \frac{a}{b}$ the water-cement ratio.

Moments about the fulcrum will determine the distance a for a given ratio, b being constant.

$\frac{a}{b}$ expresses the water-cement ratio in terms of weight.

The machine consists of two parts. The rigid-frame and the balance-frame. A knife-edged fulcrum connects the two.

The balance-frame, (bent as illustrated in order to lower the level of the hopper to obviate high lift of the cement bags) holds at one end the cement hopper and at the other the water tank. The hopper is pivoted so as to allow the cement to be dumped by tipping and to insure that the center of gravity always passes through the same point.

The water tank slides in the other end. Two screws operate the movement of it so as to allow for change of ratio.

The flow of water to the tank is automatically cut off by the falling of the trip lever (after it has first been raised to open the valve) the moment the point of balance is reached.

The indicator scale on the side of the balance-frame will tell the position of the tank for a definite delivery of water. From the gallon-scale inside the tank its position can be checked by actual calibration.

To use the machine the operator should set the tank at the position which will give (automatically) the previously calculated amount of water for the desired strength of the concrete. The machine does not have to be altered again until such time as a change in the strength of concrete is desired.

The man at the cement now does the following:

First, he empties into the hopper the required number of bags. This done,

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* Bulletin, Portland Cement Association.

Second, he raises the trip lever until it catches on the pin holding the valve in the open position and,

Third, he waits until the water has shut off *of its own accord.*

The cement and water are now dumped and discharged as soon as the whole charge is ready for the mixer. The rest is now repetition and the cement man will be found always to be ready in advance of those handling the aggregate.

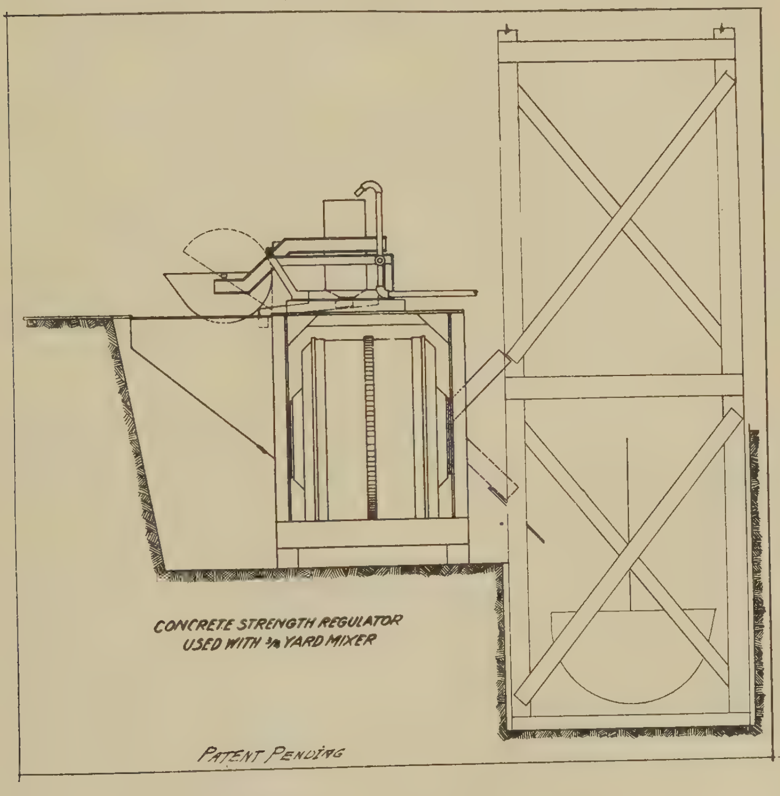

FIG. 17.—RELATION OF REGULATOR AND MIXER.

The relative position of the regulator to the mixer is shown very clearly by the sectional drawing and photograph of a machine on an actual building operation.

The engineers on all building operations where a regulator was used have approved its use as assuring them that the concrete placed in the structures was kept under rigid control.

Appendix B.

TESTING DIFFERENT AGGREGATES ON JOB NO. 199, OCT. 16, 1925.

44 BATCHES IN TWO-HOUR RUN.

	Batch No	Cylinder No.	Mixing Time, secs.	Weight Aggregate, loose weight, lb. per cu. ft.	Field Mix	Fineness Modulus	Per Cent Moisture by Weight	Water in Tank, gals.	Water-Cement Ratio	Actual 7-day Strength, lb. per sq. in.	Equivalent 28-day Strength, lb. per sq. in.	Average of Batch, 28-day equivalent	Actual 28-day Strength, lb. per sq. in.	Average of Batch, 28-day actual	7-Day Mortar Test, Computed 28-day	Average of Batch, Mortar Test, 28-day equivalent
Group I. Aggregate "G"	3	57	45	115	3:17½	5.73	2.3	18	1.05	671	1450	....	....	....	3490	....
	"	58	"	"	"	"	"	"	"	....	....	....	1480	....	3405	....
	"	59	"	"	"	"	"	"	"	666	1441	....	....	....	3405	....
	"	60	"	"	"	"	"	"	"	586	1312	....	....	....	....	....
	"	61	"	"	"	"	"	"	"	....	....	1401	1485	1482	....	3433
	5	62	45	115	3:17½	5.73	2.3	18	1.05	765	1595	....	....	....	3590	....
	"	63	"	"	"	"	"	"	"	732	1545	....	....	....	3245	....
	"	64	"	"	"	"	"	"	"	....	....	....	1810	....	3305	....
	"	65	"	"	"	"	"	"	"	683	1468	....	....	....	....	....
	"	66	"	"	"	"	"	"	"	....	....	1536	1600	1705	....	3347
	8	67	—	115	3:18.8	5.73	2.3	17⅞	1.06	805	1657	....	....	....	3180	....
	"	68	—	"	"	"	"	"	"	....	....	....	1860	....	3520	....
	"	69	—	"	"	"	"	"	"	770	1604	....	....	....	3235	....
	"	70	—	"	"	"	"	"	"	785	1626	....	....	....	....	....
	"	71	—	"	"	"	"	"	"	....	....	1629	2000	1930	....	3312
Group II. Aggregate "U"	12	72	45	113	3:17½	4.88	3.8	14⅛	0.98	1385	2490	....	....	....	3340	....
	"	73	"	"	"	"	"	"	"	....	....	....	2420	....	3310	....
	"	74	"	"	"	"	"	"	"	1350	2452	....	....	....	3330	....
	"	75	"	"	"	"	"	"	"	1190	2225	....	....	....	....	....
	"	76	"	"	"	"	"	"	"	....	....	2389	1970	2190	....	3327
	15	77	40	113	3:17½	4.88	3.8	14½	0.98	1080	2067	....	....	....	3150	....
	"	78	"	"	"	"	"	"	"	....	....	....	2190	....	3365	....
	"	79	"	"	"	"	"	"	"	1267	2335	....	....	....	3430	....
	"	80	"	"	"	"	"	"	"	1018	1975	....	....	....	....	....
	"	81	"	"	"	"	"	"	"	....	....	2126	2330	2260	....	3315
	18	82	45	113	3:15.8	4.88	3.8	14½	0.95	1356	2460	....	....	....	3340	....
	"	83	"	"	"	"	"	"	"	....	....	....	2490	....	3475	....
	"	84	"	"	"	"	"	"	"	1286	2361	....	....	....	3112	....
	"	85	"	"	"	"	"	"	"	1306	2390	....	....	....	....	....
	"	86	"	"	"	"	"	"	"	....	....	2404	2470	2440	....	3309
Group III. Aggregate "F"	21	87	20	112	3:15.8	5.44	3.6	16	1.05	1343	2443	....	....	....	3630	....
	"	88	"	"	"	"	"	"	"	....	....	....	2890	....	3825	....
	"	89	"	"	"	"	"	"	"	1202	2242	....	....	....	3685	....
	"	90	"	"	"	"	"	"	"	1244	2303	....	....	....	....	....
	"	91	"	"	"	"	"	"	"	....	....	2329	2900	2895	....	3713
	23	92	45	112	3:15.8	5.44	3.6	16	1.05	1000	1950	....	....	....	3400	....
	"	93	"	"	"	"	"	"	"	....	....	....	2840	....	3170	....
	"	94	"	"	"	"	"	"	"	1112	2112	....	....	....	3360	....
	"	95	"	"	"	"	"	"	"	1062	2050	....	....	....	....	....
	"	96	"	"	"	"	"	"	"	....	....	2071	2660	2750	....	3310
	25	97	55	112	3:17½	5.44	3.6	16¼	1.09	1290	2367	....	....	....	3045	....
	"	98	"	"	"	"	"	"	"	....	....	....	2630	....	3455	....
	"	99	"	"	"	"	"	"	"	1242	2300	....	....	....	3230	....
	"	100	"	"	"	"	"	"	"	1203	2243	....	....	....	....	....
	"	101	"	"	"	"	"	"	"	....	....	2303	2820	2725	....	3243

Direction of Prof. Krefeld of Columbia.

Recording of Mixer, Ahlers and Ehni. Sampling and Cylinders, Bowen and Morgan.

Appendix B.—(*Continued*)

TESTING DIFFERENT AGGREGATES ON JOB NO. 199, OCT. 16, 1925.

44 BATCHES IN TWO-HOUR RUN.

	Batch No.	Cylinder No.	Mixing Time, secs.	Weight Aggregate, loose Weight, lb. per cu. ft.	Field Mix	Fineness Modulus	Per Cent Moisture by Weight	Water in Tank, gals.	Water-Cement Ratio	Actual 7-day Strength, lb. per sq. in.	Equivalent 28-day Strength, lb. per sq in.	Average of Batch, 28-day equivalent	Actual 28-day Strength, lb. per. sq. in.	Average of Batch, 28-day actual	7-Day Mortar Test, Computed 28-day	Average of Batch, Mortar Test, 28-day equivalent
Group IV. Aggregate "P"	28	102	35	112	3:17½	5.23	2.8	17	1.03	1083	2071	....	....	....	3420	....
	"	103	"	"	"	"	"	"	"	....	....	....	2240	....	3200	....
	"	104	"	"	"	"	"	"	"	1144	2160	....	....	....	3460	....
	"	105	"	"	"	"	"	"	"	1146	2160	....	....	....	....	....
	"	106	"	"	"	"	"	"	"	....	....	....	2440	....	....	....
												2115		2340		3360
	29	107	45	112	3:17½	5.23	2.8	17	1.03	1363	2470	....	....	....	3965	....
	"	108	"	"	"	"	"	"	"	1293	2375	....	....	....	3950	....
	"	109	"	"	"	"	"	"	"	1403	2527	....	....	....	3545	....
												2475				3820
	42	110	60	112	3:15.8	5.23	2.8	16	0.96	1475	2626	....	....	....	3585	....
	"	111	"	"	"	"	"	"	"	1330	2424	....	....	....	3245	....
	"	112	"	"	"	"	"	"	"	1363	2472	....	....	....	3365	....
												2507				3398
	44	113	45	112	3:18.8	5.23	2.8	19½	1.16	811	1663	....	....	....	3365	....
	"	114	"	"	"	"	"	"	"	1040	2007	....	....	....	3365	....
	"	115	"	"	"	"	"	"	"	1058	2033	....	....	....	3605	....
												1901				3445

Direction of Prof. Krefeld of Columbia.
Recording of Mixer, Ahlers and Ehni. Sampling and Cylinders, Bowen and Morgan.

Appendix C.

COMPARISON BETWEEN 7 AND 28-DAY CYLINDERS—TESTS OF OCT. 16, 1925.

Batch Number	Actual 7-Day Strengths, lb. per sq. in.	Computed 28-Day Strengths, lb. per sq. in.	Actual 28-Day Strengths, lb. per sq. in.	Per Cent Lower than Actual 28-Day Strengths	Per Cent Higher than Actual 28-Day Strengths
3	671	1450	1480	2.0	....
3	666	1441	1485	7.5	....
3	586	1312			
5	765	1595	1810	15.2	....
5	732	1545			
5	683	1468	1600	12.1	....
8	805	1657	1860	9.2	....
8	770	1604	2000	16.7	....
8	785	1626			
12	1385	2490	2420	....	2.8
12	1350	2452	1970	....	15.7
12	1190	2225			
15	1080	2067	2190	5.6	....
15	1267	2335	2330	7.5	....
15	1018	1975			
18	1356	2460	2490	1.2	....
18	1286	2361	2470	3.8	....
18	1306	2390			
21	1343	2443	2890	15.4	....
21	1202	2242	2900	21.6	....
21	1244	2303			
23	1000	1950	2840	31.2	....
23	1112	2112	2660	21.7	....
23	1062	2050			
25	1290	2367	2630	10.0	....
25	1242	2300	2820	19.4	....
25	1203	2243			
28	1083	2071	2240	7.6	....
28	1144	2160	2440	11.5	....
28	1146	2160			
Average	1059	2028	2276	12.0	9.2

7-day and 28-day cylinders were made out of same samplerful. Bracketed results indicate that the average of these was used for comparison with 28-day results.

DISCUSSION.

Mr. Shaffer. IVAN O. SHAFFER (*By Letter*).—We note in Mr. Ahler's paper that we are quoted as having endorsed the so-called concrete strength regulator, the use of which we permitted on the National Metal Etching Co. building built under our supervision by Barney-Ahlers Co. We are desirous of stating our experiences in connection with the use of this device on the operation referred to above and explain certain statements in the paper which we deem need elucidation.

All of the tests of which we are aware and referred to in the paper as pertaining to the National Metal Etching job were made on Oct. 16 at which time at least 50 per cent of the concrete work had been installed. The results of these tests did not reach us until the concrete work was practically completed. The tests therefore were not conducted for the purpose of determining proper proportions for the work.

Periodical tests were not to our knowledge made although contemplated by the contractor at the beginning of the job, and in the absence of such tests we are unable to endorse the apparatus as one giving uniform concrete. As a matter of fact the amount of honeycombing was far in excess of what we consider allowable. We repeatedly took exception to the amount and quality of the pre-mixed aggregate used which criticism was justified by the results obtained and the subsequent test results.

The test results, copy of which we append hereto, show the Kittanning Mix to give the lowest results and was the mix used in the greater part of the work and was only changed after repeated requests on our part.

In operating the concrete plant described in the paper, the procedure was to shovel aggregate into the mixer hopper, no means being provided to regulate the amount going in each batch which necessitated constant supervision on the part of our inspector.

Our opinion regarding the strength regulator is that no better control of the water-cement ratio will result from its use than from any well-constructed water-measuring appliance such as any calibrated container.

As to the method of introducing an indeterminate amount of aggregate, particularly pre-mixed aggregate, this does not meet with our approval for reasons stated above.

Strength of concrete is not the only criterion of quality and porous and honeycombed concrete while it may have the requisite strength is not desirable, and these qualities can only be obtained by careful measurements of the aggregate.

Regarding the curves published in the paper, we wish to call attention to the fact that approximately 50 per cent of all the data are based on the tests, copies of which are attached hereto, which were conducted on the National Metal Etching Co. building.

Report of the 7 Day Compression Test Cylinders made on October 16th 1925 at the National Metal Etching Building — Long Island City

Using Different Aggregates with Alpha Cement
Water controlled by the Ahlers Concrete
Strength Regulator

Group No. 1 Kittanning Mixture

Batch No	Cyl. No	Mixing Time secs.	Weight Aggreg Loose Wet lbs p. cu ft.	Field Mix	F.M.	Percent of Moist by Weight	Water in Tank Gals	W/C	Actual Strength lbs p. sq. in	Equiv. 28 Day Strength lbs. p. sq. in	Average Strength of each Group 28 Day equiv.	7 day mortar Test with Cement Sample	Average strength of each group of Mortar Tests at 28 day equiv.
3	57	45	115	3:17½	5.73	2.3	18	1.05	671	1450		2110	
"	59	"	"	"	"	"	"	"	666	1441		2050	
"	60	"	"	"	"	"	"	"	586	1312	1401	2050	3585
5	62	45	115	3:17⅗	5.73	2.3	18	1.05	765	1595		2190	
"	63	"	"	"	"	"	"	"	732	1545		1930	
"	65	"	"	"	"	"	"	"	683	1468	1536	1975	3532
8	67		115	3:18.8	5.73	2.3	17⅞	1.06	805	1657		1880	
"	69		"	"		"	"	"	770	1604		2145	
"	70		"	"		"	"	"	785	1626	1629	1925	3472

Group No 2 Nassau Mixture

Batch No	Cyl. No	Mixing Time Secs.	Weight Aggreg Loose Wet lbs p. cu ft.	Field Mix	F.M.	Percent of Moist. by Weight	Water in Tank Gals.	W/C	Actual Strength lbs. p. sq. in	Equiv. 28 day Strength lbs. p. sq. in.	Aver. strength of each batch 28 day. equiv.	7 Day Mortar Test with Cement Sample	Aver. strength of each group of Mortar Tests at 28 day equiv.
12	72	45	113	3:17½	4.88	3.8	14⅛	.98	1358	2490		2000	
"	74	"	"	"	"	"	"	"	1350	2452		1980	
"	75	"	"	"	"	"	"	"	1190	2225	2389	1995	3482
15	77	40	113	3:17½	4.88	3.8	14½	.98	1080	2067		1860	
"	79	"	"	"	"	"	"	"	1267	2335		2020	
"	80	"	"	"	"	"	"	"	1018	1975	2126	2070	3470
18	82	45	113	3:15.8	4.88	3.8	14½	.95	1356	2460		2000	
"	84	"	"	"	"	"	"	"	1286	2361		2105	
"	85	"	"	"	"	"	"	"	1306	2390	2404	1832	3463

Group No 3 Rossoff Mixture

Batch No	Cyl. No	Mixing Time Secs.	Weight Aggreg Loose Wet lbs p.cu.ft	Field Mix.	F.M.	Percent of Moist by Weight	Water in Tank Gals.	W/C	Actual Strength lbs.p.sq.in	Equiv. 28 Day Strength lbs p.sq.in.	Average Strength of each batch 28 day equiv.	7 day mortar Test with cement Sample	Average Strength of each group of Mortar Tests at 28 day equiv.
21	87	20	112	3:15.8	5.44	3.6	16	1.05	1343	2443		2220	
"	89	"	"	"	"	"	"	"	1202	2242		2370	
"	90	"	"	"	"	"	"		1244	2303	2329	2265	3877
23	92	45	112	3:15.8	5.44	3.6	16	1.05	1000	1950		2045	
"	94	"	"	"	"	"	"	"	1112	2112		1875	
"	95	"	"	"	"	"	"	"	1062	2050	2071	2015	3460
25	97	55	112	3:17.5	5.44	3.6	16¼	1.09	1290	2367		1782	
"	99	"	"	"	"	"	"	"	1242	2303		2085	
"	100	"	"	"	"	"	"	"	1203	2243	2303	1920	3390

Group No 4 ½ Kittanning and ½ Nassau Mixtures.

Batch No	Cyl. No	Mixing Time Secs.	Weight Aggreg Loose Wt lbs p cu ft	Field Mix.	F.M.	Percent of Moist by Weight	Water in Tank Gals.	W/C	Actual Strength lbs.p.sq.in	Equiv. 28 Day Strength lbs p.sq.in	Average Strength of each batch 28 day equiv	7 day mortar test with cement sample	Average Strength of each group of Mortar Tests at 28 day equiv
28	102	35	112	3:17½	5.23	2.8	17	1.03	1083	2071		2060	
"	104	"	"	"	"	"	"	"	1144	2160		1870	
"	105	"	"	"	"	"	"	"	1146	2160	2115	2090	3497
29	107	45	112	3:17½	5.23	2.8	17	1.03	1363	2470		2475	
"	108	"	"	"	"	"	"	"	1295	2375		2465	
"	109	"	"	"	"	"	"	"	1403	2527	2475	2155	3985

Groups Nos 5 & 6 ½ Kit. & ½ Nas. Estimated Str. 2500# & 1500#

Batch No	Cyl. No	Mixing Time Secs.	Weight Aggreg Loose Wet lbs p.cu.ft	Field Mix	F.M.	Percent of Moist by Weight	Water in Tank Gals.	W/C	Actual Strength lbs.p.sq.in.	Equiv. 28 Day Strength lbs p.sq.in.	Average Strength of each batch 28 day equiv.	7 day mortar test with cement sample	Average Strength of each group of Mortar Tests at 28 day equiv
42	110	60	112	3:15.8	5.23	2.8	16	.96	1475	2626		2185	
"	111	"	"	"	"	"	"	"	1330	2424		1930	
"	112	"	"	"	"	"	"	"	1363	2472	2507	2020	3553
44	113	45	112	3:18.8	5.23	2.8	19½	1.16	811	1663		2020	
"	114	"	"	"	"	"	"	"	1040	2007		2020	
"	115	"	"	"	"	"	"	"	1058	2033	1901	2200	3600

The curves Figs. 13 and 14 showing variations of strength due to cement are not conclusive for the reasons that the aggregate for the different groups is not the same and if all points are plotted the parallel variation is no longer apparent. Mr. Shaffer.

For the reasons outlined above we feel that we must take exception to the last paragraph of the paper quoting us with others as approving the concrete strength regulator and consequently the method of concrete making of which it is a part.

The strength regulator is not necessary for the measurement of water and the method is too elastic for proper operation without undue supervision.

JOHN G. AHLERS (*By Letter*).—Mr. Shaffer's discussion was not read at the convention and has been referred to me for an answer. It is perhaps unfortunate that since my paper was prepared and sent in for preprinting a very serious controversy has arisen between Mr. Shaffer's firm and the Barney-Ahlers Co. over the design of a retaining wall which cracked and bulged when the earth fill was placed against it, and the matter may go to litigation. Mr. Ahlers.

It is a fact, however, that periodical tests apart from the special test on Oct. 16 were made throughout the job and Mr. Shaffer speaks of the knowledge of these at the end of his third paragraph. These test cylinders broken at Columbia University by Professor Krefeld indicated an average strength above the requirements of the building department's 2,000 lb. at 28 days, even though the engineers allowed an estimated strength of 1,800 lb. at 28 days.

The special test of Oct. 16 was made for the purpose of finding the values of various aggregates and obtaining some data on the variables in connection with control of concrete. These variables are always present in all concrete, but the special test on Oct. 16 was made so as to obtain more information about these and see if they were less under properly controlled conditions.

I thoroughly agree with Mr. Shaffer in at least one thing and that is that a special device such as the concrete strength regulator is not necessary for properly controlling the water-cement theory if careful methods are used.

There was no intention to present my paper for the purpose of having endorsement of any specific tool or equipment such as the concrete strength regulator. The sole intention was to advance the knowledge of designing and controlling concrete in the field and for the discussion of interesting data and records coming under my observation during the past year.

As to specific items such as the number of diagrams from one job and Mr. Shaffer's evident intention of inferring that 50 per cent were from the job which lacks the endorsement of the engineers and therefore detracts from the value of the paper, this to my mind rather adds to than detracts from it, for only five out of the thirteen original diagrams were from the building Mr. Shaffer designed. Every one of these five was used to illus-

Mr Ahlers. trate and discuss the variables entering into concrete, so would not detract from the value of the procedure and specifications recommended, nor the conclusions reached.

As to conclusions regarding inferior quality of the work by referring to honeycombing there are no records in the writer's possession that indicate any unusual honeycombing. To the writer's knowledge there was no honeycombing found in any of the columns and the ceilings were unusually good.

I agree that strength is not the only criterion of quality but the Joint Committee has made it the criterion for concrete design. Hence the value of any contribution to the knowledge of controlling the strength of concrete. The Joint Committee has based all its calculations on shear, bond and working compression values as being direct factors or percentages of the ultimate value of concrete in compression.

I do agree, however, that it will be desirable to establish by field investigation the relation and effect of water-ratio design on the question of porosity and hardness, etc., and it is my intention to go into this during the coming year.

Prof. Duff A. Abrams, in his paper "Studies of Bond Between Concrete and Steel," *Engineering World*, March, 1926, shows that the bond of concrete to steel is a direct function of compressive strength, and states further that: "Other tests have shown that the same statement applies in general to resistance to wear, modulus of elasticity, impermeability and resistance to destructive agencies such as weather, sea and sulphate waters," etc.

I also agree with Mr. Shaffer that proper supervision is needed on concrete where definite strengths are required and it was my recommendation in discussing this paper that more young men be trained for this specific purpose.

Mr. Smith. G. A. SMITH (*By Letter*).—Mr. Ahlers stresses the point that, on any job with the same materials, the strength of the concrete is dependent on the water-cement ratio, so long as the mix is plastic. Also that due to variations in the measuring of the cement and water there might be a variation in the unit strength of about 300 lb. This is not an alarming variation in light of what is secured under field conditions, unless we are considering very lean mixtures where the water-cement ratio is high. But where we obtain strengths that vary from 1800 to 2500 lb. (Fig. 4—$w/c = 1.10$) a variation of nearly 40 per cent of the smaller strength, and such conditions are not infrequent, it would appear that there is some other factor entering into our manufacture of concrete.

The trouble is not so much with the assumption that the water-cement ratio is an indication of the strength as it is with the actual manufacturing of the concrete. The variations in strength cited are not at all unusual and, I believe, due very largely to a lack of workability. By workability I mean those desirable properties of a freshly-mixed batch of concrete which permit of its being easily handled and deposited without segregation to form a homogeneous mass.

Mr. Smith.

Mr. Ahlers points out the need in practice of using additional sand or finer sand to improve the surface appearance and the ease of working and that such variations in the aggregates will not affect the strength if the water ratio is unchanged.

We need workability in concrete and I feel that its importance should be stressed far more than ever and any means of securing improved workability should be welcomed, as long as it does not impair the strength.

I do not agree with the statement that the addition of foreign admixtures is apt to decrease the strength. If we are not sidetracked from our assumption that the water-cement ratio is an indication of strength, practically all tests made with powdered admixtures, so long as they are chemically noninjurious, indicate an increased strength for the same mix and the same water content.

It has been my experience that the use of certain powdered admixtures does materially improve the workability of a concrete and tends to increase the uniformity of the concrete mass generally, and I believe that they will help us to elminate some of the variability in strength frequently encountered.

Several mentions have been made of workability in concrete but no one has been able to define the property or tell us how it may be measured. The apparatus described by Messrs. Pearson and Hitchcock in 1923 and 1924, is, I believe, the only apparatus that gives us an indication of the relative workability of two mixes. From my experience with the test I am convinced that it deserves considerable merit but it does not tell the whole story.

With the importance of workability becoming more and more evident in the art of building concrete structures, I feel that if we would spend a part of our time, if only a small part, in discussing this subject along with strength control, we would probably be of very great service not only to the contractor but also to the owner who is vitally interested in having the best structure possible for the investment made.

I believe that if we were to take the matter seriously it would not be a great while before some one of our members would devise some means of measuring the workability of a concrete mix.

Suggestions on the Decorative Use of Concrete.

By David C. Allison.*

In the great new structural system represented by the steel and concrete frame, there has appeared a thing completely new under the sun in architecture. No structural innovation more radical has appeared before; none that at all compares in the rapidity of its growth or the extent and development of its use from the first small steel building of some thirty years ago, to the huge and varied structures that now soar into the clouds and cover acres of ground.

The social and economic changes attendant upon our rapid growth have been admirably met structurally by architects and engineers, and the public has been given fine solutions of their problems from the standpoint of efficiency and equipment. Much discussion has gone on constantly regarding the aesthetic treatment of this architecture, and while no final solution has yet been arrived at in clothing the steel or concrete forms, yet the urge and necessity to go ahead and build them by the hundreds and thousands has been constantly with us.

It is perhaps a natural thing that the impulse of the men first confronted with architecturally treating the steel frame of many stories should have been to try to adapt traditional architectural forms and motifs, especially as, to the last generation or two, a new availability to these forms has been vouchsafed through the media of publications, of photographs, and travel, as never before; but while thirty or forty years of this kind of attempt has developed many beautiful individual buildings, yet there has been an increasing consciousness that, in applying to these great blocks the Greek and Roman colonnaded bases, the huge overhanging corbeled cornices, and manifesting throughout the structure a willingness to make it look like something that it isn't, we have fallen only into a maze of structural contradiction with small artistic compensation. Yet, something like the first glimmer of hope for a better expression seems at last to be appearing on the horizon.

The new zoning and setback ordinances, now prevailing in New York, Chicago, and elsewhere, has placed in the discard this old store-box office building and forced us to begin our design instead with a towering mass, growing up out of the ground to a limited height, receding and building on up in varying planes, a three dimensional silhouette of such possibilities as are sure to encourage a freer use of the imaginative faculties in the ornamentation, and liberate us soon from much of the banality of the past.

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*Allison and Allison, Architects, Los Angeles.

When one considers the dominant characteristics of the noblest architecture of the world, he immediately appreciates the importance of this mass, silhouette and skyline. In these elements alone, irrespective of the architectural vernacular of their adornment, reside the essential appeal of a building. These new zoning laws force upon us at the outset a tremendous advantage in this matter, and are jolting us for the first time into a more intelligent study of the problem.

Reinforced concrete has been steadily increasing in the volume and variety of its uses for the past twenty-five or thirty years, and its possibilities for use in building aspiring to a more developed or finished character are only beginning to be appreciated. Architects have pretty largely assumed that if a building is to aspire to any architectural importance whatever, if anything aside from most material considerations, such as mere strength and durability, are to enter at all, the building must at least have a skin of a more aristocratic nature.

The fortunate older sisters of reinforced concrete, such as stone, marble, granite and terra cotta, have pretty generally been called into the front parlor to meet the guest, and the more humble maiden has been assigned to the duties of the scullery and asked to do only the most common and hard manual labor. She has never been thought of as being at all in a class with her sisters decoratively, or as possessing the essentials warranting her to hope even for any aesthetic equality or respect in the household of materials. We all are ready to admit that she has a vigor, a strength, a dependability and a constancy that are excelled by none of the others. The thing we have not realized is that she responds just as readily, almost humanly, to a little attention, a little kindness, and a little loving, as do her sisters.

We have, as a matter of course, for centuries past spent unlimited energy in working, carving, beautifying these other materials. We have considered them a vehicle for our finest artistic expression and have greatly respected them as such. Concrete, however, we have hesitated to handle more gently or more intimately than could be done by means of a wheelbarrow, a shovel and mixer. The one thing most needed is for more architects of designing ability to realize that this material is capable of unlimited development; it can be molded into any form that the imagination can conceive, knit into the very fibre of a structure—an integral, homogeneous part of it, and may frankly be brought clear through to the surface and admit its identity, honestly, convincingly, beautifully. If we are willing to spend but a fraction of the cost of carving and working granite, stone and marble, upon the building of plaster moulds or in ornamenting surfaces with scraffito, or stucco in its many forms, absolutely any degree of architectural richness desired may be attained, and that at a cost very much less than in any other material of like permanency.

My attention was first drawn to the fact that good-looking architecture can be built of monolithic concrete, columns, curtain walls, *et al.*,

something like eight or ten years ago, when a building, known as the Bible Institute, a thirteen-story structure consisting of an auditorium seating well on to five thousand people, and with several hundred sleeping rooms, was built in Los Angeles. This building was treated quite richly and freely in the matter of ornamentation, balconies, parapets and the like, by the extensive use of plaster molds as forms. It was well designed and a most interesting building architecturally. When the forms were stripped, the texture and color and whole appearance of the building were so lovely that it was obviously a great pity to plaster it at all, it being a particularly good job of concrete. The general excellence of its appearance came as quite a surprise to me for, in common with others, I had never thought of the material as being suitable for other than warehouse and factory construction. The owners of this building, however, wound up by covering it with a surface of perfectly smooth plaster and painting it pure white, emasculating it of much of the fine vigor and natural texture it had originally possessed. This was done over the protest of the designer of the building, John T. Vawter, who as an expert engineer as well as architect, has since gone far in the thoughtful and artistic use of this material of concrete. I am indebted to him for many of the suggestions in this paper.

The success of this building, together with the fact of its economy, and the difficulty of getting steel at the time, led us to adopt a similar construction in the University Club, a seven-story building of about 1,500,000 cu. ft., built about six years ago. This structure is practically devoid of ornamentation except at the entrance and first-story street front, where we used a facing of wet-mix cast stone, a method producing not only the best-looking but also the toughest cast stone I have seen, the air pockets giving a pleasing texture similar to the tufas of Italy. Much discussion was had with the building committee over the degree of texture that should be retained in the walls of the superstructure, but as the whole design of the building was free and picturesque, they were finally convinced that a treatment that would retain the integrity of the concrete itself, even showing the form marks, was appropriate. The argument was advanced by some that such crudity was better adapted to a roundhouse than to a university club and some felt that it should be smoothed up like a stiff shirt front. However, we eventually used one light dash coat of cement stucco, thrown on to the concrete with a brush and so thin as to allow practically all the form marks and irregularities of the wall to show through. The result was pleasing and quite satisfactory to all after it was on.

In this building also we first attempted the use of stains on cement floors, lining the surface of the floors of the larger rooms off in squares of about 14 in. and then staining them to accord with the color scheme and rugs used. The beauty and variety of color possible was quite amazing. The process consisted of two or three brush applications of a thin mineral hardener, which carried the color into the surface from a

sixteenth to an eighth of an inch, and any color in the gamut of browns, reds, greens and buffs was possible to obtain; each square being treated individually, gave us absolute control of the variation in color. These floors are waxed and polished from time to time and after five or six years of wear have taken on a patine, depth and richness of color that are surprising. The treatment cost, at the time, 10 cents a square foot, and the transformation from an ordinary gray cement floor to one similar to rich old tiling, suitable to the use of oriental rugs, is most gratifying with a low appropriation.

In this building also, where we had a number of large rooms with none too much money to develop them, we gave considerable study to the design of ceilings in concrete, dispensing with the furring and allowing the supporting slab construction of girders and beams to count architecturally from the room. These beams and girders were sized and painted in thin stains, much as the old wooden ceilings of France and Italy were painted. While they have much the appearance of wooden ceilings, owing to the impress of grain and saw marks from the form lumber, yet no attempt was made to imitate wood. We have had no difficulty from alkalies or other cement action, and the rather elaborate stencil and painted enrichment grows softer and better with age, and certainly gains much in quality from the freehand textured nature of the material.

A similar use of concrete has since been made in the buildings of the Friday Morning Club and Women's Athletic Club of Los Angeles, where in both instances the effort was first made to secure a good concrete job by the use of shiplap lumber in forms, holding all the panels horizontal, securing rather true, sharp corners, with careful placing of material to avoid undue gravel pockets, and the concrete runs being carried up in uniform stages,-a similar light dash coat of cement stucco only being applied to the surface.

On the latter of these two buildings, the structure costing about $1,000,000, a rather expensive use of scraffito was made at the street front. This material was easily and rapidly put on the surface of the concrete, at a cost of about $6 a square yard. It was soon demonstrable that absolutely anything in the way of color was possible, and in design pattern the richest and most intricate conventionalized foliage forms could be used. We employed four different colors of plaster in the working out of ornamental panels, friezes and pilasters, in tones of buffs, browns, blues and greens, harmonizing with the general stucco tone of the concrete enclosing them. The use of wet-mix cast stone was made also in the first-story street fronts.

In the Wilshire Boulevard Congregational Church recently completed, reinforced concrete was used throughout, with no cast stone except for the enrichment of the entrance doors and occasional colonnettes. This building has a tower 140 ft. high, which carries considerable surface enrichment, made easily possible by the use of plaster molds and by the simple nailing on of blocks in various patterns on the inside of the forms before concrete was poured.

There is an undoubted virtue and sense of security, in a country where earthquakes pay us an occasional visit, to have such structures as towers free from the usual doweled-on finials, wired cornices and the like, the possible scaling off of which cause an architect restless nights at times.

We found it very easy to go as far as we liked in the way of enriching the gable copings and other decorated features of the building, taking the usual precautions for excellence of workmanship. The building was given only one brush coat of stucco and retains all of the qualities of ruggedness, strength and native intergrity of the material.

We have under construction also a Christian Science church, seating 1,200, and another large denominational church costing upwards of $1,000,000, in which we are practically eliminating the use of cast stone, confining the enrichment to monolithically cast concrete so far as possible, aiming at a legitimate, straightforward use of the material.

A brush coat of cement stucco is to be used on each of these jobs, for the purpose principally of better controlling the final color, although we have here in the West cements of such lightness in tone and such general pleasing quality in color, that with some thought in the selection of sand and conglomerates, this stucco coat could also be eliminated. It has a virtue, however, in giving an added seal to the surface of the wall, softening the occasional gravel pockets and other abrasions incident to construction.

A most interesting building now nearing completion is the Los Angeles Public Library, the last important work from the hand of Bertram Goodhue. It is a monolithic concrete structure throughout, with stucco exterior, and with the principal rooms developed with stained decorative ceilings directly on the concrete.

The decorative treatment of concrete may, for convenience of description, be separated into the following divisions:

The treatment of flat surfaces in color; the treatment of comparatively flat surfaces by means of varied textures or textures and color combined; the treatment of surfaces in greater or lesser degrees of relief or full modeling.

Cement plaster on concrete has become so common and, when properly applied, it is so much a part of its structural base that, for present purposes, the two materials may be considered as identical. Rough casting of concrete has its peculiar charm of surface and the manipulation of cement plaster runs a grand scale in variety of texture, but to haggle over the relative merits of plastered and unplastered surfaces is as reasonable as to divorce the glazes from the pottery of which it is a part.

The mere painting of flat concrete surfaces, regardless of the elaboration of the processes, carries with it nothing distinctive of the material itself, and while beautiful color effects have been obtained in this way, there seems to be an ever-present danger of allowing the process to degenerate into imitation. The temptation to imitate wood is, of course,

a natural one where rough lumber has been used as forms, since the grain of the wood is generally imprinted upon the concrete. A flat surface decoration in stains, rather than in opaque pigment, need be no less pleasing or brilliant and yet may be used in a manner to heighten rather than to disguise the characteristics of concrete. A great variety of such stained treatments is possible and might be described under two headings:

First, those resulting from what is often colorless chemicals or chemical combinations; and second, those resulting from the use of finely divided pigments.

Materials of the first class may be used either in the liquid concrete as it is cast or may be carried into the material after it is dry by means of some penetrating liquid or by means of the liquid chemical itself. Materials of the second class may be incorporated with the wet concrete or may be carried in by means of any of the so-called hardeners or other penetrating liquid. In the application of any of these methods a knowledge of chemistry is essential in order to guard against the introduction of agents which might act destructively on the concrete.

Flat surfaces decorated by means of varied textures are common in the plastered walls of northern Italy and the designs run from the most simple geometric divisions of the surface to graceful representation of natural plant forms.

The impulse to scratch a newly plastered wall before it has entirely hardened is, it would seem, shared by every member of the human race from the cradle to the grave. There is something irresistibly inviting about it. If the results of such scratching off of white plaster were to reveal the warm brown tones of a soft brick wall beneath it, the art of scraffito in two colors has been discovered, and if such a brick wall had previously been covered with a coat of smoke blackened plaster before the white had been applied, the range of color would have been enlarged.

Hard scraffito is the result of applying successive layers or coats of different color plaster to a wall and, after it has taken its set, the design is scratched through in different parts to the color best suited to its representation. It will be noticed that the hard process, therefore, employs as its means a combination of relief, texture and color, and the skillful artist takes advantage of all such means by arranging the sequence of color coats to assist the effect of deep cutting.

The process as outlined has been followed for centuries and when scraffito is used to-day it is still followed. No particular advantages have as yet been taken of the use of modern machinery or the qualities of modern materials in an attempt to improve the results or conserve the time and energy of the artist. Such efforts were not necessary in the days when the designer and artisan were one, but to-day we must handle the problem of conveying to the mind of the artisan the desire of the designer, and must recognize the economic changes in the labor situation if we are to get on at all. The centuries through which scraffito has had its development have never witnessed the variety, brilliancy, durability,

FIG. 1.—UNIVERSITY CLUB OF LOS ANGELES.

In this building the character of the material of which it is built is pre-eminent, harmonizing well with the solidity displayed by the architecture.

or workability of colored plasters such as is offered for the purpose to-day, nor have the ages ever before offered a machine capable of spreading smooth surfaces of uniform thickness of such materials. In conjunction with such mechanical means of preparing the working surface, we are now also in possession of a magnificent means of chipping, scratching and cutting the surface. The means referred to is the cement gun, which handles not only cement but gypsum also with equally good results. The surface once prepared, we now have recourse to the modern pneumatic hammer or tool for cutting and surfacing and hard granular material. By means of these two present-day devices and modern colored plastic materials the staging is up and nothing is lacking but the "Designing Mind" to add a new chapter to the history of hard scraffito.

Soft scraffito is accomplished by a more direct method than the hard, for the drawing and painting are done simultaneously and both during the process of laying the background surface. It is a method of placing plaster on a wall in certain definite areas which constitute the design to be executed. It would be seen that in manipulation and method soft scraffito and fresco are identical; the difference lies only in the degree of realism attempted by the designer, in the degree of skill manifest, and in the amount of pictorial modeling employed. Wet plasters, when laid side by side, offer the same opportunity of blending or being drawn together as do oil paints on a canvas, nor is there any less possibility of the use of intermediate or joining tones or hues. By this quality of the materials it is at once seen how the skillful scraffito artist may be drawn into the realm of the true pictorial delineator and the results of his efforts culminate in fresco.

Backed by only a limited number of experiments, it seems safe to predict that, in the execution of soft scraffito, no fewer advantages may be secured from modern machinery and materials than have been pointed out in the description of the hard method. Stencils cut from light roofing materials or sheet metal withstand the sand blast of the cement gun for a considerable length of time and by means of a series of such stencils, carefully studied for overlapping, successive layers of different colored plasters may be built up into a design. Hand plastering over or through such stencils is equally effective, and when the artisan is not a designer, the expedient affords a mechanical means of executing the will of the artist to a high degree. Aside from the fact that two colors are never overlapped to form a third, the process and ingenuity of designing the stencils is identical to that of block printing.

Related to both methods of scraffito and its monochrome ancestor, is the incised ornament exemplified by the floor of the Cathedral of Sienna. It is believed that these white marble slabs were covered with wax, into the soft surface of which the lines were drawn with a metal tool, the real incision having been made by the use of acid held in place by sand or sawdust. The whole process was probably identical to that of etching a copper plate, except that problems of size had to be dealt with. In modern

FIG. 2.—LOUNGE IN THE UNIVERSITY CLUB OF LOS ANGELES.

Simple fresco decoration of the structural beams and girders, with lightly textured walls lend a soothing quality to this room.

times, concrete floors, by reason of their chemical composition, would lend themselves to the process as readily as slabs of marble. Problems enough would remain to make the work attractive, but there is no doubt that a great field of intensely interesting experiments remain to be performed along this line.

Another method of incised ornamentation consists in bringing the designer in contact with a freshly laid and finished concrete floor or wall and, after having provided him with suitable scaffolding, induce him to etch the surface after the method of dry-point etching; that is, merely scratching his design into the yielding surface with a suitable metal tool. The counterpart of the dry-point burr is present in this process and, as in the etching, must be partly or wholly removed after the surface is hard set. Fillers composed of earthy pigment and wax or cement may be rubbed into the lines if the work is intended to carry to any distance or if, for sake of cleanliness, a smooth surface is desired.

Another division of the subject deals with the methods of treating concrete surfaces in low relief, either pattern or matrix; raised or sunken. Since we are most interested here in those methods which are possible yet uncommon, we shall not stop to consider further the use of glue and plaster molds. These methods are established and rapidly developing into a highly technical art, both in the production of all sorts of surface embellishment in poured concrete and of cast stone in pieces to be assembled later. The tendency at present toward cast stone rather than toward plaster forms for monolithic casting is regrettable, but it seems safe to predict that when once we have regained our equilibrium from the art stone tilt which we are now experiencing, we shall settle down to a more legitimate use of that worthy material as inserts for monolithic casting.

Cast stone, however, is not the only material which lends itself to a legitimate use as inertia—tile and wrought and cast metal of all kinds are at the disposal of the inventive designer.

Wood formwork readily lends itself to the construction of rectangular and geometric matrix panels for monolithic castings, but little has been done toward the further embellishment of such panels with anything resembling the free flowing or curved lines of natural form which, by long inheritance, we have come to regard as essential to architectural adornment.

Present needs are for a plastic material with which one may freely model his ornament on the inner face of wood panel forms. The material must be easily worked like clay, but unlike clay it must possess the quality of not drying, cracking, shrinking and loosening itself from the wood form in the process of hardening. Preferably it should be a material of considerable strength, yet capable of being dissolved or softened in order to remove it from deep grooves or undercuttings. Experiments have been made with mixtures of clay, sand and glycerine; with sawdust and glue; with sand, water and flour; in fact all of the materials known to foundry practice may be looked upon as promising possibilities. The fact that the

FIG. 3.—WILSHIRE BOULEVARD CONGREGATIONAL CHURCH.

Here ornamentation is limited to those points of the structure first met by the eye—the entrance facade and the imposing heights of the campanile. The simple architectural lines effectively create the dignity sought.

FIG. 4.—WILSHIRE BOULEVARD CONGREGATIONAL CHURCH.

This surface shows clearly the character of the substance of which the structure is wrought—no effort having been made to conceal the markings of the forms.

FIG. 5.—WOMAN'S ATHLETIC CLUB, LOS ANGELES.

A cement-wash, of cream color, covers the monolithic concrete face, the scraffito work and the stone-marked stucco on the street side being the only decoration.

FIG. 6.—WOMAN'S ATHLETIC CLUB, LOS ANGELES.

In the interior smooth plastering prevails with ornamentation in fresco on the beams and arches. The fineness of the plastered detail is noteworthy.

whole process of modeling is negative or the reverse of what is to be obtained in the finished work is apparently no great handicap. One soon becomes accustomed to think in terms of the matrix, and only a little practice is needed to overcome what at first appears to be an insurmountable difficulty. Anyone who has mastered the difficulty of reversing his drawing in the making of an etching, has accomplished a feat equal in every way to that of matrix modeling.

The possibilities for the enrichment of broad surfaces of monolithic concrete by the methods suggested above are infinite and yet we are waiting for the proper inventive designer to develop and make it available to the architect.

There are also opportunities for direct first-hand modeling in concrete itself. By carefully selecting the materials and proportions of the mixture, a concrete may be made which lends itself to tooling without suffering any great loss in either strength or durability. With such a material and the modern pneumatic tools at our disposal, the work of a sculptor could be reduced to a minimum by an ingenuous form builder capable of blocking out or of bounding a statue by the flat planes which are already his stock in trade.

High relief, built in place, is also possible by means of the cement gun. Steel armatures are readily and economically covered by this means and the degree of finish to which the work is to be carried is limited only by the skill of the director of the nozzle. While a tooled finish of such work is possible and while a toolable mixture may be deposited by means of the cement gun, yet it is doubtful whether the modeler who has gained a degree of familiarity with the nozzle will ever consent to any such subsequent finish of his work. There is a joy in the building of masses and of shaving them into planes while quite plastic which increases as familiarity with the tool progresses, and it is highly possible that through this feeling, a freedom of expression may arise sufficient to develop a new style of architectural sculpture, bearing a similar relation to our present formal modeling that a rough watercolor sketch bears to a finished painting of fifty years ago.

While our friends of the 6H pencil have made possible, through the use of this delightfully flexible material, concrete, the support and construction of any building shapes that can be devised, it is now up to the addicts of the charcoal and 4B to realize that this is a material of the greatest architectural possibilities and one in the use of which, as they study its intelligent application, they can again build structures that will stand on their own feet architecturally, devoid of the sham and inconsistencies of much of our recently past work.

Stucco Textures and Colors.*

By O. A. Malone.†

I believe it would be of more than passing interest if I would tell you a little bit of the evolution of stucco that has taken place on the Pacific Coast. In anything I say I want to again remind you that I am talking entirely from the plasterers' standpoint.

As I first knew stucco it was applied on wood lath, very much as interior work is done. We possibly used lime mortar with a great deal of hair in it. A second coat possibly carried a little cement, and the third coat maybe all cement; rather a foolish practice, but we did it just the same, and it required time to show us that we were wrong.

The next step was the use of metal lath, using practically the same mortar. The rusting out of the metal not closed in the back, quickly showed us that we were again wrong. Our next step was the use of what we commonly know as the lime-cement mixes that are in general use today. That is about 10 per cent of lime used with the cement and about three parts of sand. This also was not a real success because this material was not thoroughly shoved through or even had a backing to receive it. It allowed the back side of the metal to be exposed. Again, after a few years, we found nothing but the brown marks of rust.

The next step was the use of open mesh. Mr. Allison once said to me: "That kind of stuff is naturally fool proof," and I hardly know a better way to put it because it would be utterly impossible to plaster that without thoroughly filling up all the space back of it before you would get anything to remain on the front of it. In other words, it simply became a knack of pouring the cement mortar in and around and in the back. Open mesh also had a weakness because we usually furred it with ⅜-in. wood lath, necessarily cutting the slab in two, or at least in thirds at those points.

Then it was that we began to see furring devices—such devices as a plain nail with a bridge that furrs and holds out the metal, at least ¼ in. from the backing. It is notched, and this head comes down and forms the loop, holding it in place. It has no tendency to weaken the slab at that point.

I am going to do what the plasterer generally calls "scratch coat" this panel. I want you to note here that this mortar is just simply "young concrete" and it is not so easily handled. The next procedure is to score this with a metal scratcher. I do not believe there is anyone here who is not thoroughly convinced that that metal in the back is thoroughly covered.

*This is a talk which accompanied a practical demonstration of placing stucco made on the platform by Mr. Malone.

†President, California Stucco Products Co., Los Angeles.

If we had the opportunity to tear off the paper on the back we would see nothing but the plain concrete after it sets.

This next surface that we are going to work over is of concrete units, which we consider to be the very best base on which to apply stucco. However, we are sure that it is necessary to apply a scratch coat and allow that scratch coat to become dry before the browning or second and the finishing coats are applied. If that is not done the likelihood is that in wet weather the joints will show through, and this is the only method I know of that will absolutely prevent that. Both the surface that is

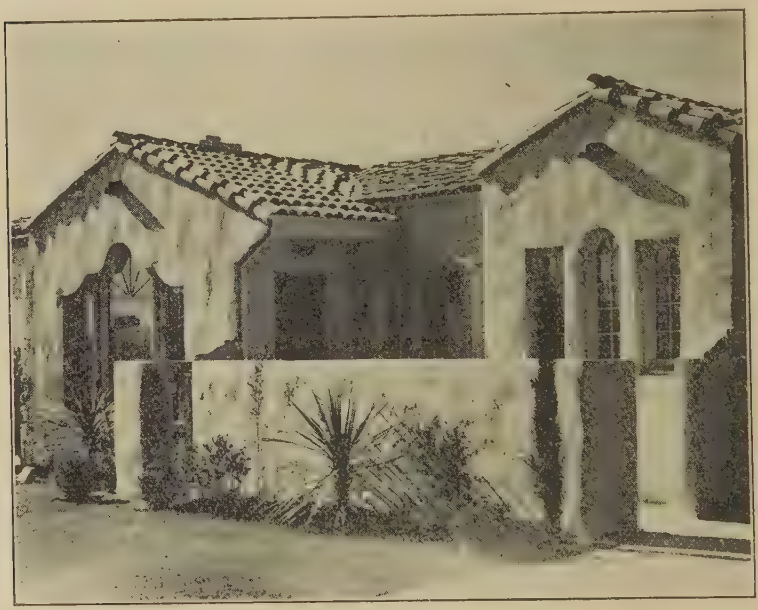

FIG. 1.—THE UNUSUAL BEAUTY IN TEXTURE AND COLOR OBTAINED HERE WITH PORTLAND-CEMENT STUCCO IS ESPECIALLY NOTICEABLE AS THE SUNLIGHT STRIKES SHADOWS ACROSS THE SURFACE.

scratched on the metal and the other surface are in the same condition. From this point on one procedure works on either: that is, a browning coat of approximately ½ in., then the surface may be rodded straight and true in every direction, or it may be left wavy and untrue, giving a wavy effect as the desired finish would suggest.

I have understood here in the east you have a great deal of fear about plastering on monolithic concrete. I want to say that that too we consider the very best base upon which we can apply stucco. If we strike a wall that seems to be a little bit smooth, we will probably make up a dash coat,

composed of one part cement and one part sand, and we will partially dash that surface. That is thrown on in a whipping motion, using a dash brush. We know without any question that the trowel as it works over a surface is absolutely enclosing about 30 per cent of air underneath it. By the force of the throw of the dashing method the little particles divide the air and strike fully on the face and become practically a unit with the wall. I would like to tell of an instance wherein a moving picture was shown before the local union of plasterers in Los Angeles. The picture was made by one of our largest gypsum plaster manufacturers, and it

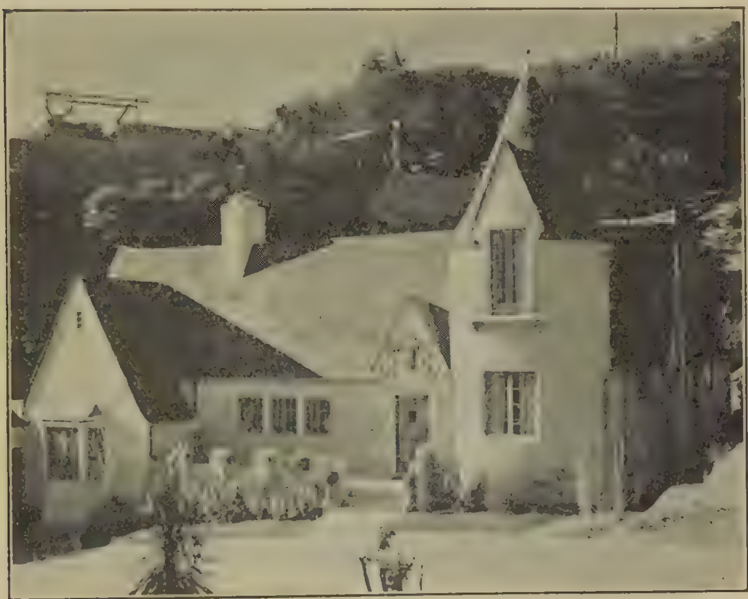

FIG. 2.—TUCKED AWAY AMID THE SLOPES OF HOLLYWOODLAND, A SUBURB OF LOS ANGELES, ARE MANY HOMES SUCH AS THIS, WITH UNIQUE TREATMENTS IN ARCHITECTURE AND SURFACING.

showed the plasterers how they should plaster different surfaces. They got along splendidly until they got to the monolithic concrete wall; then, instead of the fellow appearing in the picture with a hack and trowel, as usual, along came a fellow with a hammer and beat and hammered that wall for four or five minutes. The next thing he did was to wash it with acid, pouring the acid into wooden buckets instead of metal ones. He then carefully washed the acid from the wall and used a steel brush that was drawn carefully over the wall. After considerable patting, he finally started to plaster. There was an old plasterer sitting back in the audience

and he hollered, "Hey, Jack, wait a minute, you forgot the powder puff."

Again I remind you that I am telling you about what is only happening on the coast and in the course of the elimination of methods to apply stucco, we have also gone through an elimination of textures. We have almost completely passed through all of these rough dry efforts that are so common through other sections of the country, and we have gotten down to the more natural textures, and those without any evidence that the plasterer has tried to do something. In other words you get no feeling

FIG. 3.—A BIT OF PICTURESQUE FRENCH NORMANDY PRODUCED IN THE PRESENT-DAY SUBURBAN COMMUNITY, IN MODERN FORM BUT ACCURATELY RETAINING MANY OF THE CHARACTERISTICS OF THIS UNUSUALLY PLEASING STYLE.

that the plasterer put a bump on here and a bump on there, but one that would very much represent the free hand drawing of the artist. In accomplishing these different things we use different trowels. If we use the ordinary small trowel, we take it to an emery wheel and grind it down. If we hand it to the plasterer and tell him to plaster the wall without any further instructions, he can hardly do anything else but give you a nice, pleasing texture. We know, too, that in many of the old structures that we think we are imitating, that we may find in Mexico, Italy or Spain that those textures were done very lightly with a mason's trowel, because in

those days the man who built your foundation, built your chimney and very likely did your plastering, did not have any other tools except the mason's trowel. Consequently if we grind down the plasterer's trowel and give it to him, he will give us the same results.

I want to call attention to one thing that I believe is likely to do more against the use of stucco than any other one thing I know of; and that is that many people are in the field selling things for stucco bases which would only prove disastrous if used. I know that is true. Last winter I

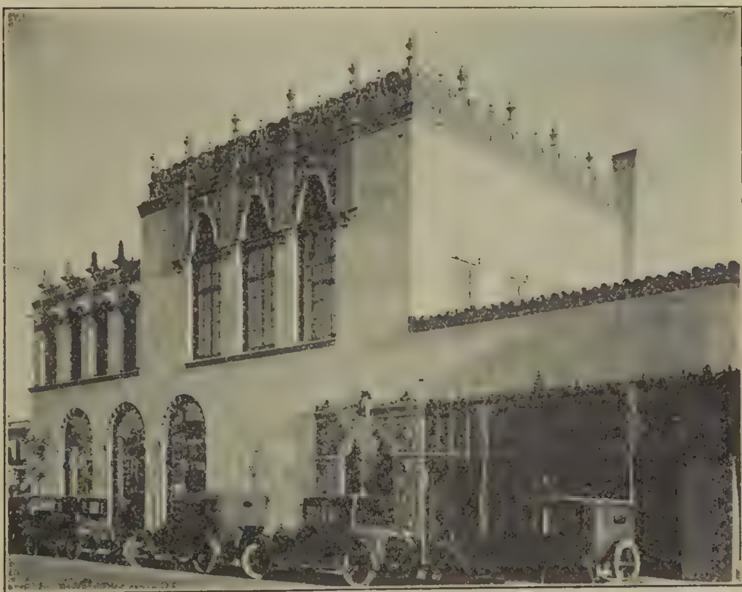

FIG. 4.—ONE OF THE EXCLUSIVE SHOPS IN THE BUSINESS DISTRICT OF LOS ANGELES, ITS COLORED STUCCO EXTERIOR PLEASINGLY TINTED AND EMBELLISHED WITH CAST CONCRETE ORNAMENTATION.

talked before the metal lath salesmen in New York City, and I tried to explain the method of this reinforced-concrete slab as thoroughly as it was possible for me to do so. After the meeting was over there were a couple of gentlemen who had waited until they got a chance to talk to me privately. They told me that they had devised and were then promoting the very thing I was talking about, and they insisted on my seeing it, and finally came to my hotel a day or two later and asked me to go over it.

They had a kind of corrugated paper with a wire running up and down through it. There has just recently come to my notice, a nail, which looks somewhat like a button or a clasp. The inventor of this nail pro-

poses, on frame construction, to sheet the building and then drive these nails 8 in. apart and stucco it. In other words he proposes to button the stucco on.

FIG. 5.—A STRIKING VIEW OF THE ELK'S TEMPLE, LOS ANGELES, AN IMPOSING STRUCTURE OF MONOLITHIC CONCRETE CONSTRUCTION FINISHED IN STUCCO—A MASTERPIECE OF THE MODERN AMERICAN STYLE OF MONUMENTAL ARCHITECTURE.

For the color demonstration I have selected three rather loud colors. By no means am I recommending them for use, but knowing that I was going to work under lamplight, the colors would hardly show after you

got them on the wall unless they were fairly strong. Another thing I do not believe I am far from the truth when I say that when we are to blend colors and do it charmingly, we must have the service of a color

FIG. 6.—A STRUCTURE OF COLOSSAL PROPORTIONS, THE NEW AL MALAIKAH SHRINE AUDITORIUM IS TYPICAL OF THE INCREASING USE OF STUCCO FINISH ON BUILDINGS OF A MONUMENTAL CHARACTER.

artist; but if we can contrive a way in which these colors can be blended without being controlled by the mind, I believe that we are reaching a result that is fully equal. I am first going to show you one method that will do that.

Mr. Hart asked me to tell you that this surface represents the browning; we scratch coated the other metal panel, and then had that become

FIG. 7.—"FITTING A HOUSE INTO THE SIDE OF A HILL" HAS HERE PROVED AN ACCOMPLISHMENT FROM AN ARCHITECTURAL STANDPOINT. THE COLORFUL SHADING OF THE STUCCO ALSO HARMONIZES WELL WITH THE SURROUNDING NATURAL TINTS.

thoroughly dry and cured, we would have applied another coat that we would call the browning or second coat. That goes on about ½ in. thick; and forms the reinforced-cement slab with the reinforcement in the center

of the slab and thoroughly encased. It is just concrete, and is the same as pouring concrete with the metal reinforcement properly placed.

Instead of diving into this and hawking it up, I am going to place the material on the hawk—"one, two three," until I have a hawk coat. Now if we cared to have the blend larger, we might have the hod carrier place the material in his hod—"one, two, three," and then just bring it up and dump it out; then we would dive into it in the usual way and hawk it up easier, but in this case I am going to put it on this way. Now we will use the round pointed trowel. Plasterers will note that the stocks

FIG. 8.—A CREAM SHADE OF STUCCO AND A RICH, DEEP GREEN IN THE ROOFING TILE EFFECTIVELY ENHANCE THE SIMPLE ARCHITECTURAL LINES OF THE AUTOMOBILE CLUB OF SOUTHERN CALIFORNIA, ONE OF THE SHOW BUILDINGS OF LOS ANGELES.

are different. That surface may be left that way. Or it may be blended just as far as you care to blend it by using a brush—this way. Very often, after the surface has hardened, we will go over it again and trowel it. If we had only one tone of color, the second troweling would create a two-tone feeling.

Another thing I want to call your attention to, is that the pores of that surface are closed, and therefore it is practically impervious to moisture. Note what happens as I make a straight up and down motion. That is just another one of the million possibilities that you have for making texture.

I know that in the use of stucco there is a very serious problem—that of crazing. I can tell you that there are millions of yards of stucco work in California that are not craze-cracked. I am going to show you the only method I know of which gives a reasonable chance of eliminating it. It is well understood that this panel has often been made with a brand of cement in which the chance of crazing is almost nil. It is the finishing coat, I believe, in which the greater amount of crazing occurs. Consequently, we virtually point the surface. We rub the coat on very thin; and the mortar I am using is a fairly fat mortar. We then grind the surface down with what the plasterer calls a carpet float; it is just the ordinary float with a piece of carpet nailed on the back of it. On the surface so treated there is not a thirty-second of an inch of mortar, and I do not believe it would ever craze.

The last tool I used was what we call a Dutch brush, pretty well worn out. Around through the country I see an attempt in almost every place to do a texture that suggests travertine, and in nearly all instances the plasterer seems to misunderstand what to do. For that reason I am going to attempt to show him what we think is a travertine texture, out on the coast. I am first going to show you what it appears to me that the plasterer does. I am speaking now of the Edgewater Beach Hotel in this city. Then he seems to have very carefully measured down another couple of inches and he does this again, and then again he takes another measure because he doesn't want to vary the least bit and he does this. When he runs the trowel over it, that is what he gets. The effect there, you will note, as the trowel runs over it.

I believe I have gone far enough to show you that there really is no end to the textures that can be laid in portland cement stucco; they are only limited by the imagination of the plasterer or those directing the work. Any number of tools may be used from the bare hand to the special tools here, or others. We know today that we can get color in stucco that is permanent, because in California I am informed that it is the only state in the Union where the cloth manufacturer will not guarantee the color of his cloth, consequently we have more colored stucco on the coast, I think, than in any other section of the country.

Concrete Building Units and the Cincinnati Building Code.

By George R. Hauser.*

Prior to the summer of 1924 the concrete building block industry in Cincinnati was struggling along rather indifferently, manufacturing blocks of a more or less unknown quality. These blocks or building units were frowned upon by the architects, builders and the interested public, who, in general, made little use of them in building construction work. The Cincinnati building code at that time contained sections designed to discourage the use of hollow concrete building units, and required foundations of such blocks to be at least 12 in. thick for an ordinary one-story frame residence, and correspondingly greater thicknesses for larger buildings. Their use was absolutely prohibited in any construction work other than dwelling houses.

During the early part of 1924 these manufacturers organized an association, and with the help of the Portland Cement Association, prevailed upon the commissioner of buildings to consider the advisability of amending the building code to the end that hollow concrete building units would be given a place in the sun and an opportunity to compete with other types of materials.

As commissioner of buildings in the city of Cincinnati the writer became convinced that a very useful purpose could be served in revising these code requirements, and after considerable study and deliberation there ensued the present Cincinnati ordinance regulating the character and use of such block or units.

While it was at first proposed to license the manufacturer of such blocks, as do similar ordinances in other cities, it was finally deemed more advisable to require that none but approved blocks be permitted in building construction work. While this ordinance has virtually the effect of the ordinary license regulation, it eliminates the legal question of the right of a city to exercise its police power by licensing certain builders or manufacturers of building material and not uniformly requiring the same of others.

This ordinance does not concern itself with the manner in which the hollow or solid concrete units are produced except Section 433-1, which qualifies the cement and aggregate used in same. The finished product is required to meet certain compression and absorption tests. These tests are generally made in one or the other of two local laboratories in Cincinnati suitably equipped with a Riehle and an Olsen testing machine, respectively.

Test Conditions.—Concrete units are tested for use either with exposed surfaces or with surfaces protected. Where surfaces of units are to be exposed to the elements the ordinance requires that the unit pass a test of

*Commissioner of Buildings, Cincinnati, Ohio.

one thousand pounds per square inch gross area, and for protected surfaces seven hundred and fifty pounds per square inch gross area.

No restriction is made as to the percentage of air core or void to solid material, nor does the ordinance fix any minimum thickness for the webs or wall of the units. Samples are taken at random and sealed with a special cheatproof device by representatives of the department of buildings and sent to the laboratory. Seven test reports selected at random from our files show an average ultimate compressive stress of 1,322 lbs. per sq. in. gross area and an average of 3.9 per cent absorption by weight. Our ordinance fixes the maximum absorption at 14 lb. per cu. ft. of actual material. As this will be found to be approximately 10 per cent by weight, the department accepts units when absorption tests show them to remain under 10 per cent by weight when so tested.

Test Supervision Necessary.—It is important that the enforcing officer or building official satisfy himself that the tests are made under proper conditions. In Cincinnati we found that when the reports from the one laboratory were not satisfactory, the manufacturer would request that his product be tested by the other, and strangely enough, the report made by the second laboratory showed the product (made in the same way and of equal age, etc.) to have met all requirements of the ordinance. The department of buildings' investigation of these variations of laboratory reports revealed that the machine used for the one test had no universal head and that the compression stresses were not being uniformly distributed over the entire unit, causing early or premature failure. On the other hand, we found that the machine used for the other test was of inadequate size and that the unit tested was necessarily permitted to project beyond each side of the seat and head of the machine, causing sheer stresses to be set up at these points and giving a very questionable result and one manifestly unfair and consequently unreliable and unacceptable for the purpose.

The department of buildings' experiences with these tests is mentioned here merely to show the importance of personal supervision by the building officials in checking up on this work. Both of the laboratories referred to are recognized as competent and trustworthy, the one being the University of Cincinnati Engineering College.

While the department is not directly concerned or interested in the proportions of sand, aggregate and cement used in the manufacture of these units, we do, however, as a matter of interest and policy, check and keep record of this phase of the work. We have found that the block or units showing the greatest strength contain a larger amount of coarse aggregate and a longer period of time is spent in the mixing. The average time period of mixing is about 2½ minutes and the average mix consists of one part cement to five parts sand and aggregate. Generally, the aggregate used varies from ⅝ in. down about 55 per cent of the total aggregate consists of clean washed sand. The average manufacturer reports 13.4 blocks (size 8 in. x 8 in. x 16 in.) produced from one 94-lb. sack of cement, and these are sold at from 18 to 21 cents per block at the manufacturer's

yard, with an added charge of 5 cents per block for stone-face finish. Recently we have noted a tendency to decrease the amount of material in each unit. While the average block eighteen months ago was about 67 per cent solid, we now find an average of about 58 per cent solid, but have noted no decrease in the ultimate strength, indicating a somewhat richer mix.

Education Needed.—The department believes that if more scientific methods were used a larger number of units could be produced from the same amount of cement. We do not believe that the concrete block manufacturers of Cincinnati are peculiar to the industry and believe they are fairly representative of their kind in any other American city. The department of buildings has noted that these manufacturers mostly do not avail themselves of the ways and means offered by scientific research laboratories to improve their products. It is a regrettable fact that although much valuable and interesting data on the subject of proper proportioning and mixing of concrete are easily obtainable, the average manufacturer does not even know that such data exist, or, if he does know, fails to apply them to his work.

That manufacturers of concrete units do not take advantage of the valuable information compiled by students of this subject is no fault of the organization mentioned, but means must be found to compel these men to take heed and be guided in their work by proper influences or their business will suffer economic extinction. What would happen to us if our average medical practitioners failed to use the latest and most proven methods in combating disease? Of what value to us, when the truths learned in the research laboratories are not put to use in an effective and beneficial manner! Here surely is a field of endeavor worth the task. Perhaps it is largely on account of these shortcomings in our ordinary builders and concrete unit manufacturers that most building codes set up requirements which often seem unreasonable and out of proportion to actual conditions.

Criticism has been made of the 1,000 lb. per sq. in. requirements for concrete units to be used in ordinary two-story residences and which will actually be stressed to only about 25 lb. per sq. in. Personally, I admit that this requirement may be excessive, but we must not fail to consider that these units are often subjected to lateral stresses from soil, frost and water pressure, and to poor workmanship, the bearing surfaces being often only 50 per cent. covered with mortar.

I believe the real cure for excessive building code requirements is better material and workmanship, and that improvements in our codes in this respect will be in proportion to improvements in the industry affected. There is always opposition to lowering the standards of requirements in building codes, no matter how unreasonable they may seem, either by conservative architects, etc., or by selfish interests. It therefore behooves the industry which would change an excessive building code requirement, first to show good faith by improving itself and show a high standard being produced by the industry generally.

DISCUSSION.

Mr. Woolson. IRA WOOLSON—From a rather intimate experience of some years with building code requirements, I am convinced that one of the greatest drawbacks with which the concrete block manufacturer has to contend is the rather severe restrictions of building codes in general as regards the recognition of walls built with such units, compared with other materials.

It is a common requirement, I think, that insurance organizations class concrete block buildings as frame. That has been the custom for some years. It is gradually being overcome, but it still exists to a large extent in different parts of the country because of the unfortunate experience which insurance organizations have had with the failure of wall concrete block when subjected to fire. The damage has been so great that, so far as their business is concerned, the loss is practically a total one and they might just as well be frame; in fact, I have heard the argument made that insurance companies would prefer frame because then there would be no cost of tearing away the rubbish.

Such experience, as all concrete block manufacturers know or ought to know now, after the investigations that have been conducted at the Underwriters' Laboratories under the joint jurisdiction of that organization and the Concrete Products Association, largely resulted from the cracking of the webs in the blocks, due to unequal expansion of the two sides of the wall, that exposed to the fire and the outside. If a large percentage of blocks in a wall are found to be cracked after a fire, although the wall may be standing, it is still unstable and does not sound right to the hammer, the owner of the building says to the insurance company: "That wall shall be made as good as it was before," and that usually means its demolition. Now that difficulty which has been inherent in concrete blocks can be largely overcome by processes of manufacture, and up-to-date, wide-awake men are doing it; and the tests that I spoke of before have demonstrated that the better grades of blocks can be made to withstand severe fire tests. When blocks having such qualifications can be assured, it is manifestly the duty of insurance rating organizations to give them a better classification than hitherto justified.

One of the warnings sounded by that report was that quartz gravel was not a suitable aggregate to be used in concrete blocks, where it was liable to be subjected to fire. I am just wondering how many manufacturers have listened to that warning? How many manufacturers are continuing to make their blocks of quartz gravel, irrespective of the location where they are to be used? If they have not listened to the warnings, they are likely to pay the penalty, for it is an undoubted fact that quartz gravel is not suited to the use of concrete that is liable to be subjected to fire, either in concrete blocks or in solid concrete construction. I speak of this rather forcibly because I was very much interested in developing that fact

a good many years ago. The tests that were made at the Underwriters' Laboratories confirmed that fact and the tests which have been made at the Bureau of Standards further confirm it. Mr. Woolson.

I think that is one of the features that this organization ought to take up. It should be emphasized in its specifications and then through its proper committee, the matter should be taken up with a suitable committee on the same subject in the American Society for Testing Materials and get the specifications exactly the same in all particulars in both organizations. Give all the latitude that should be given; reduce the ultimate strength stresses where they probably can be reduced in safety, but let us have one standard instead of two or more which vary in certain features. I believe the product is capable of much larger and better development than has occurred during my experience in the last 15 or 20 years.

There was a time when I tested a great many of these blocks, and I found they had a wide variation in quality, and I understand that that variation still exists to a greater extent than it ought to, after the investigations that have been made and should have tended to correct those defects.

E. W. DIENHART—In order that the record of discussion may be more or less clear on this, there is a point with regard to the salvage value or fire resistance factor of hollow building units that should be brought out. The tests made at the Underwriters Laboratories were on 8-in. hollow walls. Salvage value and fire protection are relative factors. Tests made by the Bureau of Standards on solid brick walls state emphatically that an 8-in. solid brick wall cannot be expected to have a salvage value. Economics has found a place for 8-in. masonry walls. When compared with a solid brick wall we have just as good a wall as solid brick. Mr. Dienhart.

With regard to Mr. Woolson's statement on the use of quartz aggregate, I believe he made it clear and we should have it emphatic, that the same risk involved with hollow masonry units is also encountered when that material is used in monolithic construction.

Professor Woolson touched upon another detail that is important in our industry and that is: Variation of quality. Variation of quality does not come alone from lack of attention on the part of the manufacturer, but the building code requirements of this country tend to cause a variation in quality. For instance, in our own plant, we ship concrete building tile to Detroit, where a gross area compressive test of 700 lb. per sq. in. is required. We ship to Jackson, Michigan, where 800 lb. per sq. in. is required, and we ship down into Ohio where the general requirements are 1,000 lb. per sq. in. You can readily see that some work must be done to standardize building codes so that we who ship products into these various markets will have a uniform standard of quality to work to.

MR. WOOLSON—Referring to a remark that Mr. Dienhart made that there was great necessity for the revision of building codes in order to make them uniform in their requirements, I want to say that there is no question but that it is highly desirable, and the suggestion which I made that Mr. Woolson.

Mr. Woolson. this organization get together with the American Society for Testing Materials and get a standard ordinance was aimed at the accomplishment of that purpose.

After these two organizations have decided upon what a standard ordinance should be for the complete control or use of blocks, then building code committees throughout the country have something to accept and incorporate into their ordinances, the same as they have at the present time a standard specification for portland cement. Nearly every building code in the country accepts the standard specification for portland cement because it is recognized as a standard by competent authorities. Now when your specifications are equally standardized for blocks, they will be put into the building codes in the same way and then you will have one requirement to work to instead of two or three.

Mr. Hatt. W. K. HATT—There is one phase of the subject that I think is interesting, that is the matter of keeping antiquated codes in print. I suppose a building code is an extension of the police power directed to the preservation of public health and safety, but since codes are written by engineers and by architects, they have come to be very largely manuals of construction. I understand also that in most cases it requires the publication in some local paper three times of any new code provision, which is a very expensive matter. If we could, by some means, take out of the building code all those matters of good practice, of relative economy, small differentials of improvement, etc., and publish them as a supplement or appendix, and put in the code itself only those matters which protect a man from being killed or injured by a falling building or having his health injured by lack of proper sanitary conveniences, taking care of access and exit requirements, codes might be printed in a small volume and leave to the building department, publication, as an appendix, advice as to how best to make materials stand up to the requirements.

In discussing this matter with Mr. Burton, who is commissioner of the city of Detroit, he said that was all very well and might have obtained in European cities where an architect was held personally responsible for the safety of his structure. In this country we cannot do that, we are all friendly and juries are friendly and so we have to have these supplementary manuals of instruction to get good material, good design and practice.

However that may be, I think there is an opportunity of improving codes to diminish the volume of the publication and not deal too much with the matter of good practice, because when you do that you have this contest between different producing interests which sometimes prevents an improvement of codes and makes it more difficult to advance them.

Concrete Units in Building Codes.

By Frank P. Cartwright.*

Building code requirements for concrete units present a field for simplification and standardization as fertile as any in the catalogue. There is a variety both of requirements and of limits enforced under these requirements, which defies complete description within the limit of a brief article, and which is entirely unjustifiable under the widest conceivable variation in conditions.

The subject divides itself naturally into a description of historical trends and present status, with reference, first, to requirements for the units themselves, and second, to their use in masonry. What requirements we now have in codes are comparatively recent, and there is a marked lag between development of the product and its recognition in code practice.

When the writer attempted to draw a comparison between prevailing building code requirements for concrete units and those obtaining some years ago, a number of representative codes of about the vintage 1900 to 1910 were secured and examined for regulations relative to such units. Practically no requirements were found worth mentioning and the purposes for which such units were allowed to be used were in general so limited that the lack of requirements governing strength and other physical characteristics was not remarkable. Concrete brick and tile at that time had scarcely been suggested and no recognition of them appears in code requirements.

Historical Trends.—Since that time, as you will doubtless know, there has been a tremendous expansion in the use of these masonry materials with which the somewhat ponderous machinery of building code revision has been in many cases unable to keep pace. To quote some figures from recent reports of the U. S. Department of Commerce concrete units were being produced in 1921 by 1,123 concerns and the total value of such production amounted to $31,768,143. In 1923, the latest year for which data are available, these figures had increased to 1,138 concerns with a total production valued at $53,602,321.

Appreciation of this development has not been general among those outside the industry and it is perhaps not surprising that code requirements have lagged behind technical progress in development of materials. A widely heralded handbook of building construction which recently appeared includes in its 1,400-odd pages eight lines on cement brick (so-called). Concrete block is more generously treated.

The Bureau of Standards made in 1921 an analysis of masonry wall requirements for dwellings not exceeding four stories in height. Of the

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135 codes examined, which were thoroughly representative, 54 made no provision for the use of hollow concrete block. Nor was this the worst of it. Thirty-nine were dated 1914 or later, and twelve, representing a population of 2,360,000, were of the vintage of 1919 to 1921. Seven of these 12, according to the latest information, are still lacking in provision for the use of concrete units.

Code Deficiencies.—To illustrate the delays and difficulties between industrial development of the material, and the enactment of corresponding code requirements, a few examples may be cited. Montgomery, Ala., in its building code of 1903, recognized the use of concrete blocks. The news had not penetrated to Florence, Ala., in 1918. Lynn, Mass., in its 1921 code provides for concrete units; Salem, Mass., ten miles away, in its code of the same year makes no mention of them. A subsequent examination of representative code requirements in January 1925 shows 10 out of 50 ordinances which do not mention concrete blocks. It is true, of course, that when a material is not mentioned in such legislation its use and development is not hampered, and to this extent such omissions may be regarded with equanimity. On the other hand, building officials have an unfortunate tendency to consider that when a material is not mentioned, its use should not be allowed. There is also the abuse to which lack of regulation exposes an otherwise creditable material.

The situation in regard to concrete brick is of course much worse, since the development of these and of suitable code requirements for them is more recent. The Department of Commerce has on file five codes, representing a population of 1,530,000 which allow free use of concrete brick, and there are at least five more, now in process of enactment, which, unless changed at the last minute, will provide for their use. They doubtless are being used much more widely, but whether under desirable regulation, from the viewpoint of both maker and purchaser, is uncertain.

Many codes which do mention concrete units are so vague in their requirements that the material is subject to any whim of interpretation. Some codes specify requirements for units, but say nothing about their use in walls. In at least one code the converse is true. A discussion of some of these shortcomings follows, based on the requirements of 50 representative codes for concrete blocks.

Strength and Absorption Variations.—Compressive strength limits for block to be laid with cells vertical vary from 750 to 1,000 lb. per sq. in. of gross area tested as laid in the wall, with an average of 870 lb. A few codes still cling to the limit based on net area, with its obvious difficulties for thin-walled block, the allowed strength being uniformly 1,500 lb. Four other codes do not state whether the limits apply to net or gross section, and considering the range of practice no conclusion can be drawn from the stress limits specified. A few ordinances allow block to be tested with cells horizontal, specifying average strength limits of 300 lb. with a minimum of 200 lb. for any of those tested. In two cases the extraordinary requirement of a tensile strength test is included.

The majority of codes specify an absorption limit, the prescribed values ranging from 5 per cent to 15 per cent of the weight of the block, and obtained by 24 to 48 hr. immersion. The favorite figure is 10 per cent. No codes are at hand except those of Washington, D. C., and Evansville, Ind., which vary this requirement to recognize block of light aggregate.

Undoubtedly, the character of available aggregates and other considerations in a given locality may make it possible to secure a stronger or less absorptive unit than in other places. There is no reason, however, from a building regulation viewpoint, for a strength requirement greater than needed for durability and safety, and such a limit will naturally be the same everywhere that block are used for the same purposes. Greater strength or durability where considered desirable by builders, should be secured by agreement with the manufacturer; not by public fiat.

A number of codes, perhaps 25 per cent of those examined, supplement strength and absorption requirements with directions for the size of aggregate and the proportions of the mix. Methods of curing and age of block when tested also come in for their share of attention. The proportions specified range from 1 cement-4 sand, to 1 cement-3 sand-5 coarse aggregate; and the maximum age when tested from 28 to 36 days. The age at which block may be used in the wall varies from 14 to 30 days though it is obvious that by steam curing, or by the use of special cements, the necessary strength could be secured much sooner.

If block are placed on a proper performance basis as to strength and absorption, there should be no need of rules governing proportions. Such rules are unenforceable in practice, and operate merely to hamper the manufacturer or to weaken the code.

Performance Fundamental Consideration.—Obviously the performance of the unit in the wall is the fundamental consideration. The unit cannot be placed in the wall until it reaches the job and if the code requirement reads that samples be taken from the job, the necessary control over strength of the unit when used in the wall will be secured without hampering time restrictions.

Other Considerations.—The branding of units by the maker, required in about one-third of the codes examined, is believed essential to effective control of the material and if accompanied by a requirement that units be dated as well as trade-marked it assists in preventing use of green block.

The freezing and thawing tests characteristic of some early code re- reason for their revival.

A majority of the codes examined restrict the percentage of hollow spaces in block and tile, the limits ranging from 25 per cent to 50 per cent of the gross section. The limits are accompanied in some places by specific requirements for thickness of shells and webs, these operating in some cases to prevent the manufacture and use of concrete tile. This is believed to be entirely out of line with the general trend toward a performance basis. Concrete tile which meet the requirements for compressive strength on gross sectional area will almost certainly have sufficient net section to

permit effective bedding in mortar and there is no good reason why such units should not be used within the ordinary height and stress limits for walls of such materials, as readily as clay hollow tile are now used. The Department of Commerce recommendations coincide with those of the Portland Cement Association and The American Concrete Institute in avoiding entirely this form of restriction.

Block Utilization Deficiencies.—Considering the variations in code requirements for concrete units it is not surprising that there should be even greater differences in those controlling the utilization of these units in masonry. As previously noted, many codes do not cover this phase. Of those which do, it is notable that codes with the lowest strength requirements for units usually permit fairly high working stresses in walls of these units. The stresses themselves range from 80 lb. per sq. in. for brick laid with cells vertical and 30 lb. for brick laid with cells horizontal, to 138 lb. without reference to method of laying. One code distinguishes between wall stresses for machine-made and hand-made block, though no such distinction appears in the strength requirements for single block. Two codes set different stress limits with respect to the mortar used. In one case a stress of 300 lb. is allowed on the net section of the block, which in effect is a much higher stress than allowed by the average of other requirements based on gross section.

The Department of Commerce Building Code Committee has given the comparative stresses for block laid with cells vertical and horizontal a great deal of study. It appears that the smaller net section characteristic of block laid with cells horizontal is fairly well offset in practice by the more thorough bedding in mortar. There is also the consideration that unless work is very carefully inspected courses with cells laid horizontally may be inserted in walls designed to be laid with cells vertical thereby defeating the increased strength which such walls might otherwise be expected to have. In view of this the same stress of 80 lb. per sq. in. gross sectional area is recommended in both cases. This should be decreased to 70 lb. where cement-lime mortar is used. Lime mortar is not recommended to be used for laying of hollow units.

Larger units are generally supposed to result in more rigid walls and in view of the comparatively generous working stress limits in the codes examined it is hard to understand why concrete block walls are limited in practically all cases to three stories or 40 ft. The two exceptions found permit their use for six-story bearing walls. The Building Code Committee, with an eye to the combined stresses ordinarily obtained in bearing walls, has increased the generally specified height to 50 ft. This is about consistent with the stress limits imposed and with the wall heights for ordinary construction determined by economic and fire resistance requirements.

Summary or intelligible comparison of thickness requirements for bearing and non-bearing walls of concrete block is practically impossible because of the multitude of variations met with. In general, the thickness

limits follow closely those imposed for solid brick walls and are somewhat greater than recommend by the Building Code Committee in its report on masonry wall construction. In two cases only the thickness required was 10 per cent less than that for solid clay brick walls. Other departures were in the direction of greater thickness.

The use of concrete block for fire walls is quite uniformly forbidden in current codes. This, however, is hardly supported by the results of more recent tests of such construction and the use of the material for this purpose has been recognized in the Department of Commerce report as follows:

Fire walls, defined as interior walls extending continuously from basement to parapet, are required to be not less than 12 in. thick for residential buildings and 16 in. for other occupancies. Fire division walls, extending from floor to floor in buildings of fire-resistive construction, may be not less than 12 in. throughout. These limits apply for both concrete block and concrete tile.

In summary, code requirements for concrete building units are in a bad way. They are well behind technical progress in the material, they are restrictive and uneconomical in some cases, they lack the uniformity necessary to the marketing of a standardized product over any considerable area and the advantages of large-scale production of the material and they merit energetic effort on the part of professional groups toward their improvement. A critical and uncomplimentary description of existing conditions is of little value unless it suggests definite measures for bringing about their betterment.

Suggestions.—With a view to providing a basis for the development of such measures, the following suggestions are offered:

1. Discrepancies between current recommendations for public regulation, under the police power, of the quality of the material should be ironed out. There is no objection to variation in the specifications recommended by different organizations for general purposes. Where these recommendations are intended to influence building code revision, they should be in agreement. Many recommendations received by the writer offer evidence of the confusion among code-drafting bodies occasioned by differences in recommendations proceeding from national organizations apparently equally authoritative. Where specifications are intended primarily for other purposes than code revision, this fact should be emphasized for the benefit of code-drafting organizations to the attention of which such specification may come.

A comparison of the requirements of the American Concrete Institute, the Department of Commerce Building Committee, the American Society for Testing Materials and the Portland Cement Association shows complete agreement on but one or two points. The average compressive strength of brick, for instance, is put at 1,500 lb. per sq. in. by all, although two specify "as laid in the wall" and two "flatwise." The A. C. I., however, recognizes three different grades of hollow block, with different compressive

strengths for each while the Portland Cement Association and the Building Code Committee name only one. The Portland Cement Association allows a unit compressive strength in masonry walls of 100 lb. per sq. in., while the Building Code Committee has not felt justified in going over 80. In absorption requirements, the A. C. I. and Building Code Committee are in agreement while the Portland Cement Association and the A. S. T. M. go their separate ways. It is obvious that efforts to remedy the existing confusion in code treatment of the subject will be much more effective if the recommendations of these organizations are in more complete agreement.

2. So far as possible, code requirements should be on a performance basis, with emphasis on performance when built into masonry rather than as single units. This means that strength requirements based on net section, thickness of shells and webs, percentage of hollow space, proportions of aggregates, methods of curing, etc., as criteria of suitability should be dropped and requirements for strength over gross area and absorption based on amount of water should be emphasized. In the field of utilization, working stresses should be kept low until it is demonstrated by tests similar to those which have been met by clay brick and other materials that higher limits are permissible. Height limits should be incorporated in building codes sufficient to encourage energetic development of the material through such tests and through study of its possibilities.

3. Further tests on concrete brick masonry should be made. There is available a very large collection of test data on the strength of clay brick masonry. The data on which recommended stresses for concrete brick masonry are predicated are fewer in number and in the interests of a more general knowledge on this subject it is desirable that further work be done. The question of whether the tests should come at the initiative of the manufacturer or of professional groups interested in the use of concrete is a question which it seems to the writer might with propriety be discussed at a meeting of this sort.

Effect of Lime on Concrete Products.

By Paul C. Cunnick.*

REPORT OF INVESTIGATIONS MADE AT THE ROCK ISLAND ARSENAL DURING 1925.

Many authorities have concluded that the use of more than 10 per cent of lime in concrete was not warranted. It has been assumed that this was also the economical limit in the manufacture of concrete products, but no data could be found on the subject. Apparently no serious investigation has been made of the use of lime in this important branch of the concrete industry. The object of the following tests was to indicate the effect of lime on concrete products and this report covers about 1500 test operations to this end.

Acknowledgment is made to Herman Meier, of the Northwest Davenport Cement Block Works, Davenport, Iowa, and to Sidney P. Moore, president of the Builders' Materials Co., Cedar Rapids, Iowa, whose plants made the test units in their regular operations; and to Col. D. M. King, commanding officer, and John Robertson, of the laboratory, Rock Island Arsenal, where the tests were made. The investigation was sponsored by the National Lime Association and their representative, J. S. Elwell, collaborated throughout.

OUTLINE OF INVESTIGATIONS.

Block Plant Test No. 1.—The first units for this test were manufactured at the plant of the Northwest Davenport Cement Block Works, Davenport, Iowa. This run consisted of 36 variables, with 6 mixes and 6 lime contents and it totaled 250 test units. Crushing tests at the age of 7 and 28 days showed the strengths to be increasing at 20 per cent addition of lime, which was the maximum used. This made necessary another plant run to determine the point where the addition of lime would cause the strengths to fall off.

Block Plant Test No. 2.—The second field run was made at the concrete block plant of the Builders' Materials Co., Cedar Rapids, Iowa. Thirty-six variables—including both hydrated lime and quick-lime putty—were run for a total of 990 test specimens. These blocks were shipped to the Rock Island Arsenal for strength and absorption tests. Five units were provided for the absorption and five for crushing at 28 days, 3 months, 6 months, 1 year and 2 years respectively, or 30 units of each kind. All

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tests except the 1- and 2-year strengths are now completed. In this test the strengths at 28 days and 3 months were so unusual that it seemed advisable to run some laboratory experiments in confirmation.

Laboratory Experiment No. 1.—Study of certain conditions and results of manufacture in these two commercial plants leads to a consideration of curing methods. A laboratory experiment was made at the Rock Island Arsenal in which 6 mixes and 6 curing methods were used for a total of 36 variables and 180 test specimens. Conditions paralleling in every possible way the Cedar Rapids run were used.

Results of this experiment based on 28-day strengths, indicate that steam curing may be eliminated by the use of lime, under certain conditions. Further study is planned.

Laboratory Experiment No. 2.—In order to confirm the results obtained in Plant Test No. 2 and Experiment No. 1, another laboratory experiment was made. This work paralleled the Cedar Rapids run in all respects; i. e., materials, mix and manufacture. All materials were obtained from the Cedar Rapids plant. Two curing methods, 3 mixes and 11 lime contents gave a total of 66 variables and 330 test specimens for this work.

All investigations made to date have been with products made by the dry method. The plant and laboratory operations have closely followed the "Recommended Practice for the Manufacture of Concrete Products" of the American Concrete Institute. Crushing and absorption tests were made as approved by the same authority.

GENERAL INDICATIONS.

Briefly summed up the general indications of the results are:

1. *Appearance* of the product is improved by all percentages of lime.

2. *Strength:*
   - (a) Up to 40 per cent by weight hydrated lime gives an average increase in strength of approximately 1 per cent for each pound of lime added per sack of cement.
   - (b) Aged lime putty gives considerable increase in strength, also much quicker set than equivalent amounts of hydrate.
   - (c) All percentages of lime tested show increase in strength from 28 days to 6 months of age.

3. *Absorption:*
   - (a) Determined by the standard immersion method absorption increased uniformly from 6.2 per cent without lime to 7½ per cent with the maximum lime content used.
   - (b) Determined by impounding water on one face, absorption is not increased by the use of lime.

4. *Permeability* as determined by impounding water on one face, is eliminated by use of 20 per cent or more of hydrated lime or equivalent in lime putty.

5. *Penetration of Dampness* into the product decreases as the lime is increased.

DETAILS OF INVESTIGATION.

Block Plant Test No. 1.—Several plants were considered for this preliminary investigation and the plant of the Northwest Davenport Cement Block Works was chosen as they were operating along the lines of good practice. The test was run by the regular operatives and care was taken

TABLE 1-A.—PROPORTIONING DATA AND TEST RESULTS BLOCK PLANT TEST NO. 1.

Mix.	Cubic Feet Natural Materials.		Cubic Feet Dry and Rodded Materials.		Fineness Modulus of Mix.	Total Water Content, pounds.	Blocks Per Sack of Cement.
	Sand.	Gravel.	Sand.	Gravel.			
1:4	2.00	3.00	1.74	3.00	4.20	49	11.0
1:5	2.25	3.50	2.83	3.50	3.90	56	13.8
1:6	3.50	4.25	3.04	4.25	4.00	63	16.6
1:7	4.00	5.00	3.48	5.00	4.02	70	19.4
1:8	4.67	5.67	4.06	5.67	4.15	78	22.2
1:9	5.17	6.33	4.50	6.33	4.01	85	25.0

7-DAY COMPRESSIVE STRENGTHS.
Pounds per sq. in. of gross area.

Mix.	No Lime.	5%—5 lb. Hydrate.	10%—10 lb. Hydrate.	15%—15 lb. Hydrate.	20%—20 lb. Hydrate.	10%—22½ lb. Quicklime Putty.
1:4	1232	1258	1490	1330	1450	1750
1:5	863	752	1040	1063	980	1110
1:6	615	480	555	659	752	885
1:7	430	632	553	581	728	664
1:8	409	450	449	530	436	640
1:9	314	343	439	427	456	....
28-DAY COMPRESSIVE STRENGTHS.						
1:4	1473	1655	1805	1551	1430	2120
1:5	996	1034	1180	1245	1066	1245
1:6	943	775	945	1040	1204	1109
1:7	585	816	707	856	957	1007
1:8	575	853	622	774	710	837
1:9	426	440	515	610	570	....

NOTE.—All strengths are the average of three crushing tests.

in the measurement of materials to insure uniformity and in recording conditions. Table 1-A gives the data of proportioning as recorded. Mixing in a Blystone mixer 1½ min. dry and 1½ min. wet was uniform in all cases. The blocks were cast in an Ideal horizontal core machine and cured in saturated air at 90 deg. F. for 48 hours and then put into open March storage. The consistency of the mix was as wet as the machine operator could reasonably handle and care was taken to insure uniformity in all operations of proportioning, mixing, casting and curing.

The blocks were tested for crushing strength at the ages of 7 and 28 days at the Rock Island Arsenal and the results are shown in Table 1-A.

TABLE 1-B.—PROPORTIONING DATA.

Determined in Block Plant Test No. 2 and used in all laboratory experiments.

PROPERTIES OF AGGREGATES.

	Sand	Gravel
Wt. of 1 cu. ft. of material in the natural state	96.9 lb.	104.2 lb.
Wt. of above material, dry and rodded	92.8 lb.	101.6 lb.
Volume of above material, dry and rodded	0.84 cu. ft.	0.94 cu. ft
Wt. of 1 cu. ft. of material, dry and rodded	110 lb.	108 lb.
Bulking of ratio of wet to dry material	119%	106%
Water content per cu. ft. material in natural state	4.1 lb.	2.6 lb.
Per cent of water by weight in natural state	4.2%	2.5%
Sieve analysis:		
Retained on a No. 4 sieve	2%	10%
8 sieve	11%	58%
16 sieve	38%	98%
30 sieve	74%	100%
48 sieve	94%	100%
100 sieve	99%	100%
Fineness modulus	3.18	4.66
Required percentage for fineness modulus of 3.80	57½%	42½%

DETAILS OF PROPORTIONING.

Mix. Cement to dry, rodded aggregates.	1 : 6	1 : 7	1 : 8
When mixed the separate vol. shrink to.	85%	85%	85%
Sum. vol. sand and gravel before mix	7.05 cu. ft.	8.22 cu. ft.	9.42 cu. ft.
Required volumes—dry and rodded:			
Sand—57½% of sum of volumes	4.05 cu. ft.	4.72 cu. ft.	5.42 cu. ft.
Gravel—42½% of sum of volumes	2.99 cu. ft.	3.48 cu. ft.	4.01 cu. ft.
Required volumes—in natural state:			
Sand—119% of dry volume	4.82 cu. ft.	5.60 cu. ft.	6.45 cu. ft.
Gravel—106% of dry volume	3.18 cu. ft.	3.68 cu. ft.	4.25 cu. ft.
Field mix as used			
Sand	4.8 cu. ft.	5.6 cu. ft.	6.4 cu. ft.
Gravel	3.2 cu. ft.	3.6 cu. ft.	4.2 cu. ft.
Cement, one sack	94 lb.	94 lb.	94 lb.
Blocks per sack of cement calculated.	17.8	20.8	23.8

RECORD OF WATER CONTENT FOR A ONE SACK MIX.

Water added, water in aggregates and water in putty, totaled in pounds.

Mix.	Without Lime.	All per cent of Hydrate.	22½ lb.—10% Putty.	45 lb.—20% Putty.	67¼ lb.—30% Putty.	90 lb.—40% Putty.	112½ lb.—50% Putty
1:6	80	80	84½	89	93½	94	102½
1:7	88	88	88½	95	101½	106	108½
1:8	105	105	109½	114	114½	115	119½

Block Plant Test No. 2.—In order to vary the manufacturing conditions the specimens for the second block test were made at the plant of the Builders' Materials Co., Cedar Rapids, Iowa. The materials used were all carried in stock and operating conditions were in no way changed except that a batch measuring box was installed which permitted the accurate measurement of materials. The prompt and complete delivery of materials to the mixer permitted a mixing time of 6 min., 3 min. dry and 3 min. wet, without slowing up the operation of the plant. It might be noted that this measuring box is still in use. The tests were run through

FIG. 1.—MEASURING BOX AND MIXER: BUILDERS' MATERIAL CO., CEDAR RAPIDS, IOWA.

by the usual operatives and considerable care was taken to insure that all batches were uniform and of the desired proportions. The consistency for each variable was as wet as could be properly handled, throughout the operation, without slump. To this end the judgment of the machine operator was of great assistance in fixing the amount of water required.

Table 1-B gives the properties of the aggregates, the details of proportioning and the total water contents. Mixing was done in a Blystone tilting mixer and this mix was conveyed automatically to the feed hopper of an Anchor automatic tamper. This machine is of the vertical stripper type and is equipped with a manually-operated feed. In casting, the

amount of tamping was as uniform as the operator could obtain. Curing was in steam at 125 deg. F. for 36 hours, followed by sprinkling night and morning for 6 days, in open May storage. Figs. 1 and 2 were taken at the plant during this run.

When about 20 days old the specimens were shipped to the Rock Island Arsenal laboratory and there placed in covered storage until needed for test. Tests have been made as follows:

FIG. 2.—ANCHOR AUTOMATIC TAMPER: BUILDERS' MATERIAL CO., CEDAR RAPIDS, IOWA.

Crushing strengths at 28 days, 3 and 6 months. (See Table 3.)

Absorption by standard immersion method. (See Table 4, Column 4.)

Absorption on one face. (See Table 4, Column 5.)

Permeability Tests: (See Table 4, Column 6.)

Penetration of Dampness into the unit. (See Table 4, Column 7.)

Laboratory Experiment No. 1.—In order to determine that lime-cement products were advantageously cured by the customary steam and moist methods, it was decided to make a curing experiment. Hydrate and putty equivalent to 20 per cent of hydrated lime were used in three mixes and five specimens of each of these six variables were cured under six different conditions. These data together with the 28-day strengths are in Table 5.

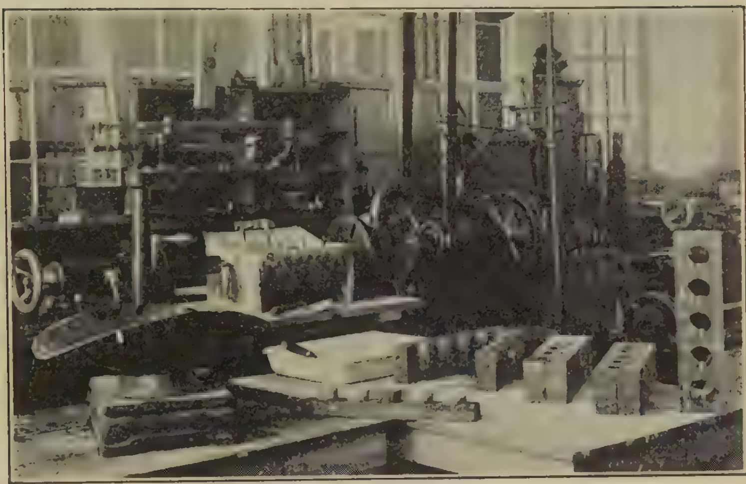

FIG. 3.—EQUIPMENT USED IN MAKING LABORATORY SPECIMENS: ROCK ISLAND ARSENAL.

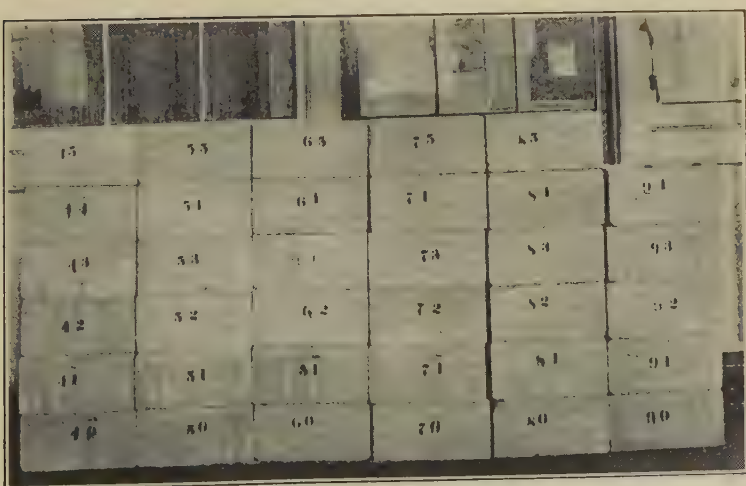

FIG. 4.—ASSEMBLED BLOCKS MADE AT NORTHWEST DAVENPORT CEMENT BLOCK WORKS.

After consideration it was decided to investigate further two of these curing methods: Twenty-four hours in steam at 125 deg. F. followed by 6 days sprinkling as compared with one week moist curing.

TABLE 2.—EFFECT OF LIME ON 28-DAY COMPRESSIVE STRENGTH.

	Marked.	Proportion of Cement to Aggregate.	Lime added per Sack of Cement, pounds.	Plant Test Number 1.		Plant Test Number 2.		Laboratory Experiment.				Average Strength as Compared With Same Mix Containing No Lime, per cent.
								Steam Cured.		Water Cured.		
				Gross Strength, pounds.	Per cent of Strength Without Lime.	Gross Strength, pounds.	Per cent of Strength Without Lime.	Net Strength, pounds.	Per cent of Strength Without Lime.	Net Strength, pounds.	Per cent of Strength Without Lime.	
Hydrated Lime Series	60	1:6	None	943	100	1130	100	1315	100	1190	100	100
	61†	1:6	10†	945	100	1235	109	1540	117	1280	108	107
	62†	1:6	20†	1204	128	1180	104	1950	148	1610	135	114
	63†	1:6	30†	....	...	1211	107	1980	150	2010	170	133
	64†	1:6	40†	....	...	1005	89	3520	270*	2800	235*	130
	65†	1:6	50†	....	...	888	79	2305	175	2160	183	129
	70	1:7	None	585	100	1016	100	1050	100	840	100	100
	71†	1:7	10†	707	123	1061	104	343	33*	239	29*	110
	72†	1:7	20†	957	162	1248	123	1080	103	1100	131	134
	73†	1:7	30†	....	...	1128	111	1520	145	1260	150	128
	74†	1:7	40†	....	...	1379	136	1425	135	1310	156	141
	75†	1:7	50†	....	...	1270	125	1365	130	1330	158	135
	80	1:8	None	575	100	663	100	705	100	715	100	100
	81†	1:8	10†	622	110	852	128	890	139	980	137	125
	82†	1:8	20†	710	121	724	109	780	111	930	130	117
	83†	1:8	30†	....	...	925	139	1157	164	1130	158	150
	84†	1:8	40†	....	...	958	144	1165	166	1365	191	161
	85†	1:8	50†	....	...	888	134	1125	157	1125	157	145
Quicklime Putty Series	60	1:6	None	943	100	1130	100	1315	100	1190	100	100
	61‡	1:6	22½‡	1109	118	1125	99	2225	171	2020	170	129
	62‡	1:6	45‡	....	...	1198	107	2015	154	2310	195	141
	63‡	1:6	67½‡	....	...	887	78	1770	135	1955	164	113
	64‡	1:6	90‡	....	...	1216	108	1710	130	2130	179	129
	65‡	1:6	112½‡	....	...	940	83	....	...	....	...	?
	70	1:7	None	585	100	1016	100	1050	100	840	100	100
	71‡	1:7	22½‡	1007	172	985	97	1380	131	1735	207	135
	72‡	1:7	45‡	....	...	943	96	1105	105	1690	201	125
	73‡	1:7	67½‡	....	...	733	74	1790	170	1705	202	130
	74‡	1:7	90‡	....	...	780	79	2005	192	1525	181	133
	75‡	1:7	112½‡	....	...	689	68	....	...	....	...	?
	80	1:8	None	575	100	663	100	705	100	715	100	100
	81‡	1:8	22½‡	837	145	756	114	1085	154	1230	172	141
	82‡	1:8	45‡	....	...	989	149	900	127	810	113¼	134
	83‡	1:8	67½‡	....	...	774	116	1240	176	1115	156	141
	84‡	1:8	90‡	....	...	617	93	1140	162	955	133	120
	85‡	1:8	112½‡	....	...	708	107	1215	172	970	135	128

NOTE.—* indicates that percentages were not used in computing averages.
† indicates hydrated lime.
‡ indicates lime putty.
All strengths are average of 5 tests.

Laboratory Experiment No. 2.—To check from another angle, the results obtained in the above tests, a series of 2 x 4-in. cylinder specimens were made in the laboratory having the same variables and materials as used in the Cedar Rapids test. The mixing was done in a small mixer

built after the manner of a standard Blystone and driven on the centers of an engine lathe (See Fig. 4). The specimens were made in a split core box of the proper dimensions. The materials were deposited in four layers and each layer tamped the same amount, to stimulate as nearly as possible

TABLE 3.—EFFECT OF LIME ON COMPRESSIVE STRENGTH AFTER 28 DAYS.
Tabulation of results of Block Test No. 2 at ages of 1, 3 and 6 months.
All strengths are average of 5 Tests.

Marked.	Proportion Cement to Aggregate.	Lime per Sack of Cement.	Strength in lb. per sq. in. Gross Area.			6 Month Strength as Per Cent of 28-Day Strength.
			Age 28 Days.	Age 3 Months.	Age 6 Months.	
60	1:6	None	1130	1086	1235	109
61†	1:6	10†	1235	1038	1350	109
62†	1:6	20†	1180	1119	1370	116
63†	1:6	30†	1211	1317	1410	116
64†	1:6	40†	1005	953	1360	135
65†	1:6	50†	888	973	1275	143
70	1:7	None	1016	839	1015	100
71†	1:7	10†	1061	1059	1115	105
72†	1:7	20†	1248	1161	1170	94
73†	1:7	30†	1128	1131	1200	106
74†	1:7	40†	1379	1195	1275	93
75†	1:7	50†	1270	973	1495	118
80	1:8	None	663	770	805	121
81†	1:8	10†	852	1043	965	112
82†	1:8	20†	724	1326	960	132
83†	1:8	30†	925	1419	1360	142
84†	1:8	40†	958	1434	1165	122
85†	1:8	50†	888	1478	1155	130
60	1:6	None	1130	1086	1235	109
61‡	1:6	22½‡	1125	1760	1585	141
62‡	1:6	45‡	1198	1629	1625	135
63‡	1:6	67½‡	887	1484	1315	148
64‡	1:6	90‡	1216	1603	1560	128
65‡	1:6	112½‡	940	1140	1180	125
70	1:7	None	1016	839	1015	100
71‡	1:7	22½‡	985	1536	1575	160
72‡	1:7	45‡	943	1508	1315	139
73‡	1:7	67½‡	733	1010	1010	150
74‡	1:7	90‡	780	1051	1070	139
75‡	1:7	112½‡	689	800	890	129
80	1:8	None	663	770	805	121
81‡	1:8	22½‡	756	779	782	103
82‡	1:8	45‡	989	1198	1175	119
83‡	1:8	67½‡	774	1023	1095	141
84‡	1:8	90‡	617	781	930	150
85‡	1:8	112½‡	708	1069	860	122

NOTE.—† indicates hydrated lime.
‡ indicates lime putty.

the casting machine. Each batch made ten specimens and was mixed 3 min. dry and 3 min. wet as in the second plant test. The cylinders were stripped vertically and moved to a rack immediately after casting. The curing methods were as noted above and storage was inside at the normal temperature for the month of August.

These specimens were prepared for test according to the standard methods as shown in Fig. 4.

Each of the strengths recorded in Table 2 is the average for five specimens broken.

TABLE 4.—EFFECT OF LIME ON ABSORPTION, PERMEABILITY AND DAMPNESS PENETRATION.

Results of Plant Block Test No. 2.

	Marked.	Proportion Cement to Aggregate.	Lime per Sack of Cement, pounds.	Absorption by 24 Hour Immersion, per cent.	12 inches of Water Impounded on Face for 24 Hours.		
					Absorption, per cent Dry Weight.	Per cent of Impervious Units.	Penetration of Dampness, inches.
Hydrated Lime Series	60	1:6	None	5.96	3.12	40	3
	61†	1:6	10†	6.55	2.65	80	2½—3
	62†	1:6	20†	6.03	2.22	100	2½
	63†	1:6	30†	7.00	2.27	100	2 —2½
	64†	1:6	40†	7.79	2.45	100	2
	65†	1:6	50†	7.25	3.09	100	1½
	70	1:7	None	6.67	3.46	20	4
	71†	1:7	10†	7.13	3.64	100	3½—4
	72†	1:7	20†	7.46	3.20	100	3 —3½
	73†	1:7	30†	7.63	3.35	100	2½
	74†	1:7	40†	5.90	3.28	100	2
	75†	1:7	50†	6.02	2.04	100	1½
	80	1:8	None	5.97	3.11	60	3½—3
	81†	1:8	10†	6.52	3.65	80	3 —3½
	82†	1:8	20†	6.57	3.78	100	3
	83†	1:8	30†	6.12	2.26	100	2½
	84†	1:8	40†	6.28	2.47	100	2 —2½
	85†	1:8	50†	7.14	3.05	80	1½
Quicklime Putty Series	60	1:6	None	5.96	3.12	40	3
	61‡	1:6	22½‡	5.60	2.21	60	2½—3
	62‡	1:6	45‡	6.25	2.31	100	2½
	63‡	1:6	67½‡	6.98	3.13	100	2
	64‡	1:6	90‡	6.87	2.33	100	1½—2
	65‡	1:6	112½‡	5.87	2.68	100	1½
	70	1:7	None	6.67	3.46	20	4
	71‡	1:7	22½‡	5.90	2.36	80	3
	72‡	1:7	45‡	6.08	2.36	100	2½
	73‡	1:7	67½‡	7.00	2.90	100	2 —2½
	74‡	1:7	90‡	6.60	2.76	100	2 —2½
	75‡	1:7	112½‡	8.37	3.58	100	1½
	80	1:8	None	5.97	3.11	60	3½—3
	81‡	1:8	22½‡	6.54	3.02	100	4 —4½
	82‡	1:8	45‡	5.96	2.65	100	3½—4
	83‡	1:8	67½‡	6.75	3.72	100	3 —4
	84‡	1:8	90‡	6.99	3.50	100	3 —3½
	85‡	1:8	112½‡	7.77	4.10	100	1½

NOTE.—† indicates hydrated lime.
‡ indicates lime putty.
All values are the average of 5 tests.

DISCUSSION OF RESULTS.

The value of any true investigation in this field is dependent to a large extent on the number of determinations made. The more values that are included, the nearer the result approaches the fact.

In this investigation into the effect of lime on concrete products there

are included three entirely different conditions of manufacture. A total of over 1,500 tests have been made. In general, every value in the tables is the average of 5 determinations and the curves shown on the plates are the average of 15 to 60 test operations.

Appearance.—No single quality of a concrete product has greater effect on marketability than appearance. A product may be sufficiently strong, thoroughly durable, and yet if its appearance is not pleasing, it will be sold with difficulty. Observation in these experiments shows that the color of the concrete is changed from a dark grey when no lime is used to a lighter grey as the lime content of the mix increases. Fig. 4, which is the assemblage of one block of each kind from Plant No. 1, shows plainly that the use of lime lightens and improves the color of the product. It was

TABLE 5.—EFFECT OF LIME ON CURING METHODS.

Tabulation of results of Laboratory Experiment No. 2.

Method of Curing.						
Steam	1 day	1 day	1 day	......	......	......
Spray	......	......	......	......	1 day	1 day
Sprinkle	6 days	20 days	......	......	20 days	6 days
Storage	21 days	7 days	27 days	28 days	7 days	21 days

Marked.	Mix.	Lime. pounds.	Crushing Strength in Pounds per sq. in., Net Area.					
62†	1:6	20†	1200	1600	685	1970	2360	3100
72†	1:7	20†	1130	1240	805	910	1695	1790
82†	1:8	20†	1310	1535	655	860	1590	1620
62‡	1:6	45‡	2360	2840	1575	1530	2400	2300
72‡	1:7	45‡	1740	1842	1500	382	750	2400
82‡	1:8	45‡	1130	1542	636	352	1180	1500

NOTE.—† indicates hydrated lime.
‡ indicates quicklime putty.

All values are the average of five specimens.

noticeable during these tests that the texture of the surface becomes finer as the percentage of lime is increased. The interior texture as observed in many fractures is much denser with lime than without. The appearance of these blocks is improved by increasing amounts of lime.

28-Day Strength.—In general, the results of this investigation show that both hydrated lime and lime putty give increased strengths to concrete products for all percentages of lime added. There are some exceptions, but the increases far exceed the decreases and in almost every case the apparent loss in strength is shown by other tests to be due to the variables of manufacture.

The 28-day strengths were determined in four tests: Two tests were made at concrete block plants and two tests were made as laboratory experiments. The two laboratory experiments are considered as one test

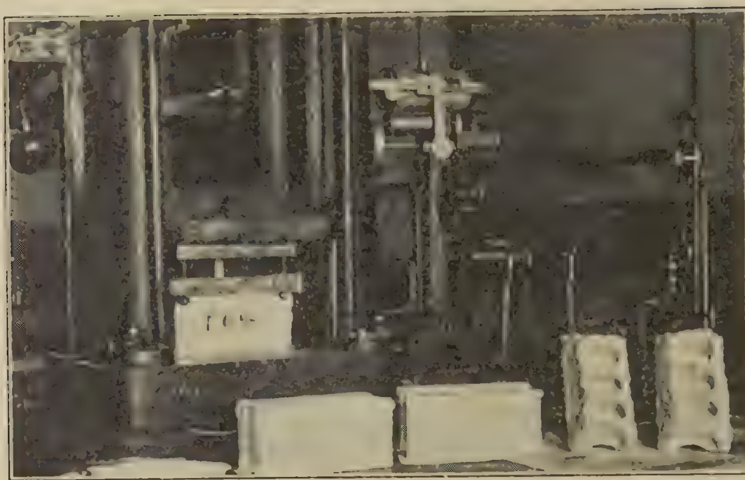

FIG. 5.—CONCRETE BLOCKS AS TESTED AT ROCK ISLAND ARSENAL.

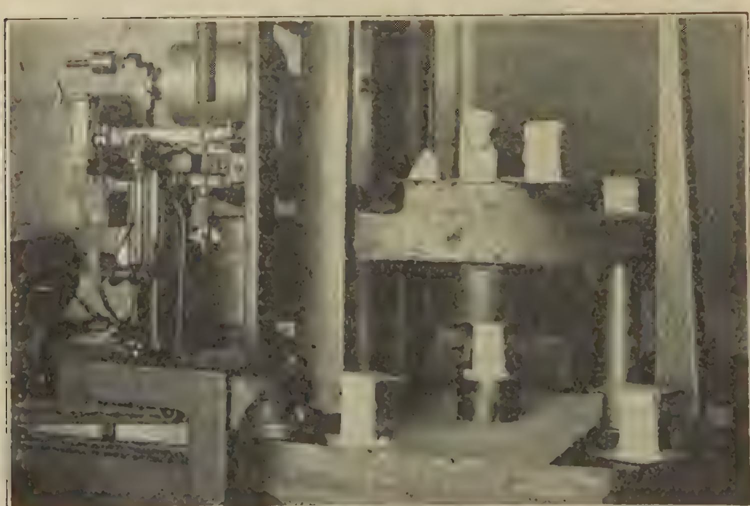

FIG. 6.—LABORATORY SPECIMENS AS TESTED: ROCK ISLAND ARSENAL.

since the specimens for both experiments were mixed in one batch. Throughout the laboratory tests the standard methods of testing were used. Fig. 5 shows the method of capping the block specimens and also

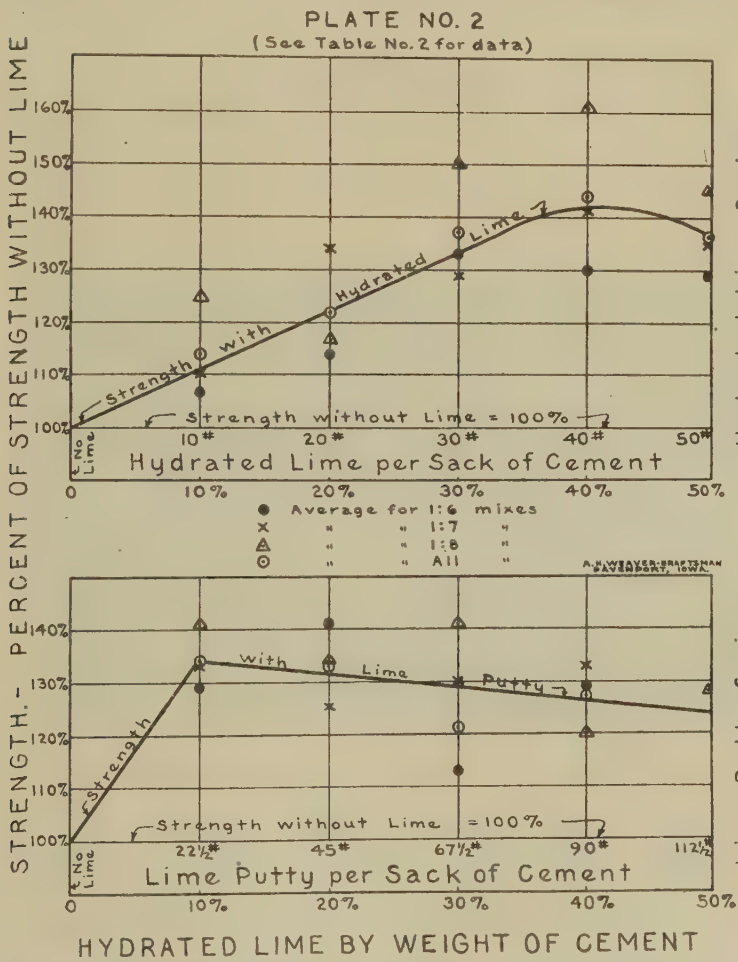

FIG. 7.—EFFECT OF LIME ON 28-DAY COMPRESSIVE STRENGTH.

several fractures obtained. Fig. 6 shows the same for the laboratory experiment.

A summary of compressive strengths, percentages of increases or decreases and average percentages of increase as compared with those without lime are given in Table 2.

Effect of Hydrated Lime on 28-Day Strength.—Fig. 7 is compiled from the average results tabulated in Table 2. The curve on this plate is the average of all tests and mixes for each variation of hydrated lime. The points of average for each mix are also identified.

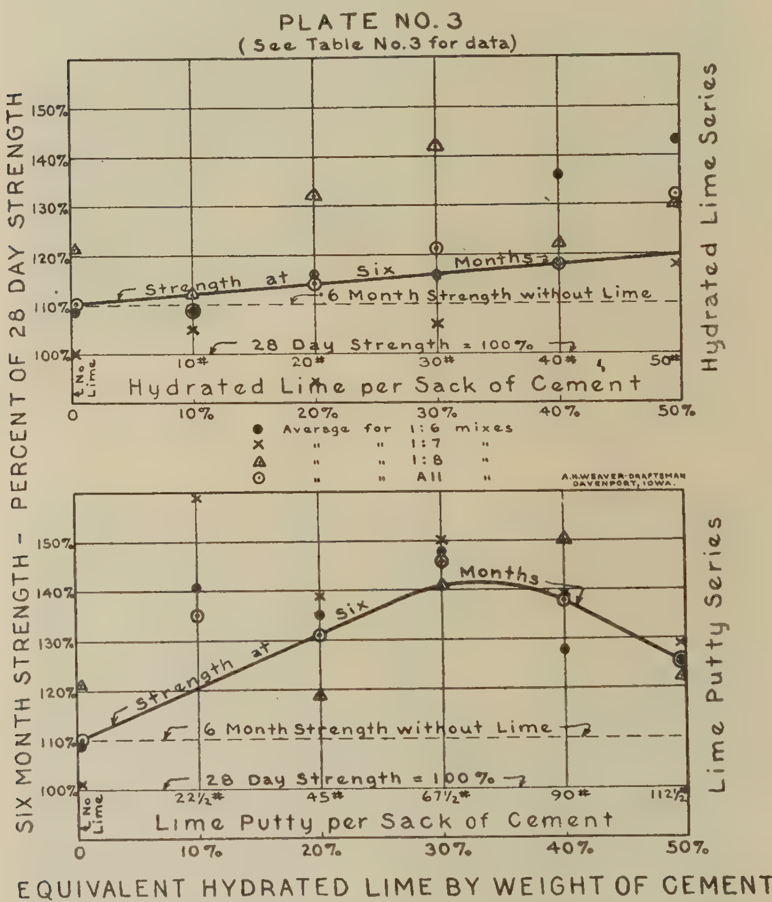

FIG. 8.—EFFECT OF LIME ON COMPRESSIVE STRENGTH AFTER 28 DAYS.

It is apparent that the strength, due to the addition of hydrated lime, increases with considerable uniformity up to 40 per cent. It is also evident that the leaner the mix, the greater is the advantage obtained by the use of hydrated lime. This last bears out the indications given in Bulletin 8 of the Lewis Institute. From these results it appears that up to 40 per

cent hydrated lime gives an increase in strength of approximately one per cent for each pound of lime added per sack of cement.

Effect of Lime Putty on 28-Day Strength.—Fig. 7 also shows a curve for the average of all tests and mixes with the variations of lime putty content.

Attention is invited to the fact that two investigations showed extremely high increases in strength when lime putty was used, while one plant test did not show as much advantage until the tests at the age of 3 months. The reason for this delayed gain in strength is not clear. Considering that the increased strength due to the use of putty was very great, even though in one case it was not attained within the time limit, it is felt that further investigation will develop material advantages

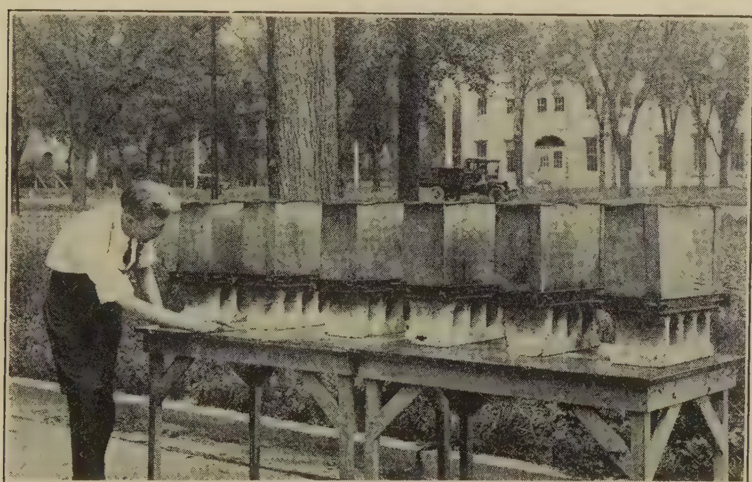

FIG. 9.—CONTAINERS FOR IMPOUNDING TESTS.

in its use. It was observed that specimens made with lime putty could have been handled to storage much sooner than otherwise. However, this can only be proved by a plant test. Aged lime putty gives considerable increase in strength; and much quicker set than equivalent amounts of hydrate.

Strength Increased with Time.—Retrogression or decrease in strength is a very serious and but rarely encountered condition in the concrete industry. This investigation would be incomplete unless it was definitely determined whether or not the addition of lime to concrete products had a weakening effect over a period of time.

With this in mind, blocks were cast in Test No. 2 for crushing at various ages from one month to two years, with extras so that the five-

year breaks could be made if deemed necessary. The results are now available for the 28-day, 3- and 6-month tests as shown in Table 3. In some cases the 3-month strengths are somewhat below those at 28 days. As this

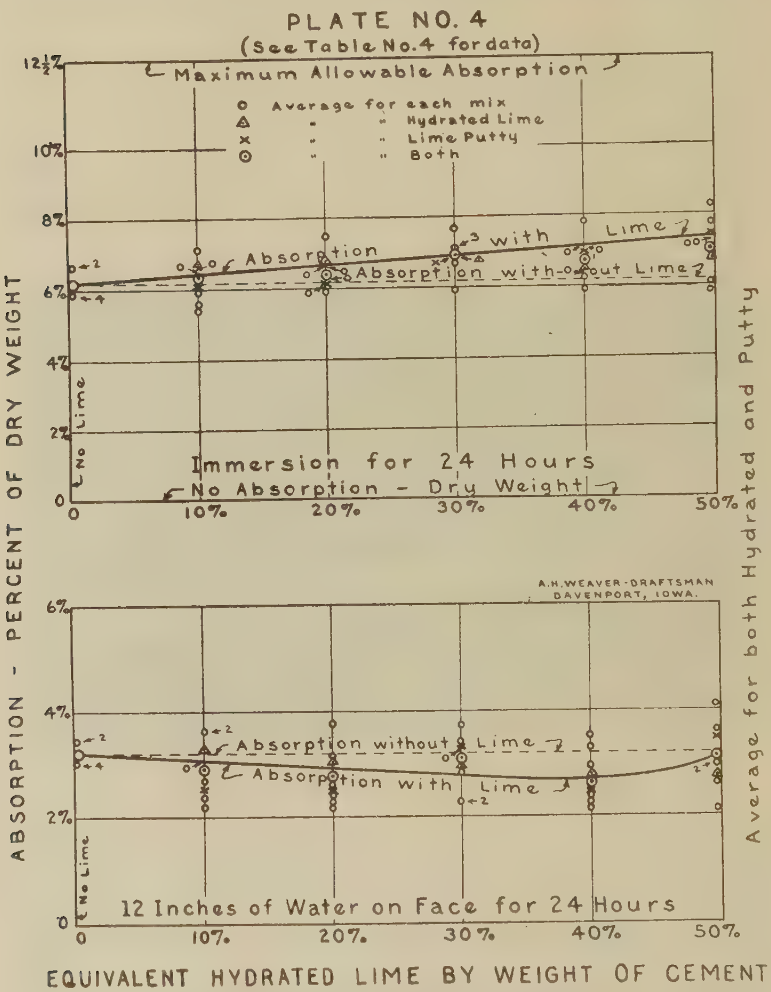

FIG. 10.—EFFECT OF LIME ON ABSORPTION.

occurs both in blocks with and without lime and is generally corrected at 6 months, it is believed that this condition is due to experimental error and uncontrolled conditions of manufacture.

The percentages of increase from 1 to 6 months are shown on Fig. 8.

It is evident that there is no retrogression. All percentages of lime show increase in strength from 28 days to 6 months of age.

Absorption by Immersion.—The standard absorption test, immersion for 24 hours, is perhaps intended to be a measure of the durability of a product. The amount of water absorbed will certainly affect durability under freezing conditions.

Absorption tests by this standard method were made and the results

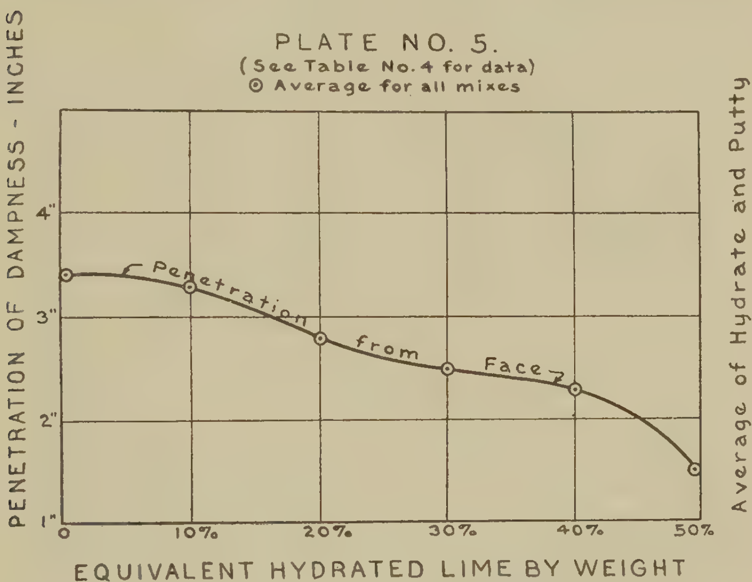

FIG. 11.—EFFECT OF LIME ON DAMPNESS PENETRATION.

are reported in Table 4. The results confirm previous work and the average increase with lime is shown on Fig. 10. Absorption determined by standard immersion method is increased by the use of lime but remains well inside the specified limit.

IMPOUNDING WATER ON ONE FACE.

In considering the standard method of establishing absorption by immersing the specimen a certain length of time and determining the per cent of absorbed water by weight, it seemed that some pertinent facts were not obtainable. Water seldom comes in contact with more than one face of a water-tight block in actual practice. A reasonable method of test that more nearly simulated this condition would be interesting. Such a test should permit the study of absorption, penetration of moisture and permeability, all of which are important. By means of a container bolted

to the specimen as shown in Fig. 9 it was possible to impound 12 in. of water on the face of a concrete block. A sponge rubber gasket makes the connection tight.

Five blocks of each of the 33 variables or 165 specimens made in the plant run at Cedar Rapids were tested in this manner.

Absorption on One Face.—The results obtained are shown in Table No. 4 and this information platted in Fig. 10 indicates that the total absorption into the exposed face is not increased by the use of lime.

Penetration of Dampness.—The depth to which dampness penetrates a specimen can be easily observed when the water is applied to one face only. In Fig. 9 Mr. Robertson, of the laboratory, is seen indicating the lines he has drawn at the maximum depth of moisture. In this photo the lime content of the blocks increases from left to right and the effect of the lime is very evident. The depth of dampness penetration is decreased as the lime content is increased. (See Table 4 and Fig. 11.)

Permeability.—The question of permeability is not touched in the standard absorption tests but in this impounding test the water flowing through a specimen can be both seen and measured. In the series thus tested, 60 per cent of the units without lime were found to permit a measurable flow of water. The addition of 10 per cent of lime resulted in 90 per cent imperviousness. Out of 120 units tested, containing 20 per cent or more of lime, only one leak was found. Permeability is apparently eliminated by 20 per cent or more of lime.

Economics of Lime in Concrete Products.—Any final discussion of the economics of this question is probably premature at this point in the investigation. The outstanding facts are that lime increases the strength of a product and improves other desirable properties. Increased strength can be utilized by obtaining a greater number of units per sack of cement when lime is used. These lime-cement units have certain advantages of appearance and water tightness that make for greater marketability.

Further plant tests will be necessary in order to determine the relation of the cost of the lime to the value of the advantages arising from its use. However, several concrete product plants have started using lime as this investigation has been developing.

DISCUSSION.

THOMAS W. NOBLE—A number of factors impress themselves on me from a study of this talk. No mention was made in the paper as to the type of aggregate used in these tests. No mention was made of one other factor; namely, whether a single brand, new brand or blended cement were used. I believe if more definite conclusions as to the advantage or disadvantage of lime are drawn that both those factors should be given weighty consideration. Mr. Noble.

It just so happens that in the past few months, my company has done a little experimenting along these lines. We were attacking it from a little different angle, in that we were interested in getting a greater machinability from our concrete. We had used hydrated lime and agricultural limestone; by that I mean limestone which is merely pulverized so that it will go through a 100-mesh screen. The results were not uniform in all cases; they followed in general the relation this shows, of little advantage in the rich mixtures and considerable advantage in the leaner and harsher mixtures.

The cement which we tried showed that the use of both materials was apparently advantageous, particularly in the case of the lean mixtures. On a fourth, quite the reverse was true. If the use of lime is merely a mechanical aid to compacting under various methods that we use in dry tamp concrete mixtures, it would seem to me that other materials should be investigated very thoroughly.

Undoubtedly a great many of the concrete block manufacturers have had the experience of using limestone screenings and getting a whole lot harder block. I know of some cases where the extreme ranges are greater than were shown in this paper. The difference between a river aggregate and a pit aggregate where a certain percentage of silt is present, is often apparent.

From the point of economy, I hope that some of the members of this organization will carry these investigations further, because I believe that cheaper materials than lime will accomplish the same result.

MR. CUNNICK—My work at the laboratory carries me into various fields. Among those is the work on steel. As everybody knows the uniformity of steel is very good. It is very difficult, even when working with so uniform a material as steel, to get your curves to all check out as you would wish, so it seems to me that our curves of yesterday and day before, the jig saw curves obtained, while they are interesting and of value, do not present as nice a picture as the curve which is properly averaged. Mr. Cunnick.

The averages on these specimens were taken to give averages as we might find them in the fields today. If you go into a man's product plant, you will find conditions in many cases rather deplorable. He is turning

Mr. Cunnick. out a block which perhaps fills the specification but the conditions he has do not always give him accurate measurements and as these test specimens were made in plants actually selling their product, I think we can attach more value to them than we could if the investigation had been carried out entirely in the laboratory, where we have laboratory conditions and know the control is better.

As far as improving the appearance, it is true that marble and the whiter materials will make a whiter block, but is the small manufacturer going to import such materials if they are not at hand to improve his block, if he can get some other substance and perhaps a cheaper substance?

The use of lime if added, I have been told, improves the color in the artificial stone up to 5 per cent. The gentleman who advises me of this fact has not carried it further, but by doing that has improved the color of his product. As the gentleman said, these are not conclusions by any means; it takes a great deal of work to bring out anything definite along such a line. This perhaps is just scratching the surface, but it is a surface well worth digging into.

Mr. Howe. HENRY J. HOWE—I have been interested recently in this matter of admixtures and replacement of the material in cement and concrete. In going over the tests made during such investigations, I have noted that all this fine material is treated either as an admixture or as a replacement material for cement. It seems to me that particularly in the case of ground limestone and other mixtures, it would be more logical to treat it as a part of the fine aggregate, particularly when the fineness modulus theory is used.

Mr. Shenk. A. B. SHENK—In Table 5, where is shown a tabulation of results on 20 lbs. of lime to the sack of cement, what is the effect of different methods of curing? Where you compare steam curing supplemented by different types of storage after steaming, such as sprinkling for a given length of time or subjecting to natural storage only, with the result you obtain, not using any steam curing to start with, but resorting only to the usual method of sprinkling and moist storage, the natural method shows considerably higher strength in all cases. Is that due only to the fact that lime is used as an admixture or would that tend to indicate that the manufacturer who is manufacturing a special product where extremely compressive strengths are desired, should stay away from any steam curing to start with, and, where possible at all by his processes, to adhere rigidly to the natural water curing?

Mr. Cunnick. MR. CUNNICK—It is impossible to make such a statement that you could stay away from your steam curing at this time. We thought it rather interesting to go into the curing end and find out which might give us the best result. The experiment was carried out during the month of August, a rather warm month, and we have naturally a warmer temperature than the normal. I am not in a position to answer the question as to whether you can stay away from steam curing.

South Water Street Improvement.

By T. A. Evans.*

Relation to Chicago Plan.—The South Water Street Improvement which, on completion, will be known as "Wacker Drive," is a major link in the Chicago Plan, being the fourth side of the so-called "Inner Quadrangle," of which Michigan Ave., Roosevelt Road and Canal St. form the other three sides. These last-mentioned streets have recently been improved or are now being improved and the South Water Street Improvement therefore becomes the connecting link in the quadrangle.

General Description.—The improvement extends from Madison St. along North Market St. to Lake St. and from there along South Water St. and River St. to Michigan Ave. In the block between Randolph and Lake St. the single level street separates into two levels, which extend to and connect with Michigan Ave. which is also of the "double deck" type.

The north-south streets that intersect the improvement will be raised so as to connect with the upper level, while a new street—Federal St.—will give access to the lower level from Lake St. Plans are also under way for widening Garland Court and for opening two new streets, Holden Court and Fork Ave., which would likewise give access to the lower level from Lake St. In addition, the two levels will be connected by means of two ramps, one between Lake and Franklin Sts. and one between Wabash Ave. and Michigan Ave.

The width of the upper level is 114 ft., except where it widens out into a broad plaza or square between State St. and Wabash Ave.

The lower level is 135 ft. wide and provides three traffic lanes for east-going traffic and three traffic lanes for west-going traffic in addition to ample space for backing up to the loading platforms along the buildings on the south side of the street. Owing to the absence of cross traffic, the capacity of the lower level will exceed that of any other street in Chicago.

A concrete dock wall resting on piles extends from Michigan Ave. to Lake St. The river front presents a series of arches which give light and air to the lower level while the edge of the viaduct is marked by a Bedford stone cornice and balustrade which, at the end of each block, adjacent to the bridges, are supplemented by a more elaborate architectural treatment of the structure involving stair cases, stone pylons and stone seats.

The improvement will be lighted by means of cluster lights on the upper level (similar to those on Michigan Ave.) and by means of recessed lights on the lower level so arranged that they illuminate the street ahead although being invisible to the driver.

*Engineer of Design, Chicago Board of Local Improvements.

Benefit to Traffic.—The improvement will benefit traffic in two ways: First, it will by-pass a large amount of traffic between the north and west sides and thus keep out of the loop district a considerable percentage of the traffic which now enters this congested area for no other purpose than to get out again; and second, it will serve as a series of grade separations, separating the north-south traffic on State St., Dearborn St., Clark St., LaSalle St., etc., from the heavy east-west traffic between the Illinois Central and Michigan Central freight yards on the east and the west side terminal district.

Benefit to Property.—Property values on South Water St. have already greatly increased because of the improvement and it is expected that this increase will parallel that experienced along the Michigan Ave. Improvement, thus justifying the claim made by the city during the long drawn out litigation which preceded the construction period and which, at times, came near to putting a stop to the entire improvement. Many new buildings have been planned for property adjacent to the improvement and one of these, the Jewelers' Building at Wabash Ave., is now under construction. The Illinois Central development east of Michigan Ave. and the Agricultural Mart planned on the site of the old C. & N. W. Ry. station are also intimately connected with the new development of South Water St.

Benefit to the City.—Apart from the direct benefit to traffic and to the owners of adjacent property, the South Water St. Improvement will be a valuable asset to the city, in that it provides a broad and beautiful thoroughfare which will undoubtedly become a popular promenade in years to come.

Construction.—Prior to the commencement of field operations a complete construction program was prepared which provided for the completion of the work by November, 1926. Ground was broken on Nov. 10, 1924. and, at the present time, we are ahead of schedule. With summer before us we should have no difficulty in completing the work ahead of schedule, barring unforeseen circumstances.

We consider this a remarkable record of construction in a congested district such as this, and it was made possible only through the successful efforts of the city administration to provide the required funds as and when necessary, and through the active co-operation of the various departments.

At the present date the structural contracts have been let along with several others for street lighting, stone work, etc. Only three minor contracts remain and these will be let shortly.

Construction has been carried on continuously from the date it was begun, maintaining practically the same speed during the winter months as in summer.

The average number of men employed has been approximately 700 daily. The work was divided in sections, each section consisting generally of one block and work has been going on almost continuously in five or six such sections at a time without seriously interfering with traffic.

There has been no interruption to elevated railroad and street car traffic and every effort has been made to facilitate access to such buildings, within the area of the improvement, as are occupied, and the almost total absence of complaints shows that we have been successful in our attempt at co-operating with the property owners and that the property owners realize that the hardships and inconvenience to which they now are put will be amply offset by the benefits which they will derive from the finished improvement.

Cost.—The improvement involves 300,000 cu. yd. of excavation, 120,000 cu. yd. of concrete, 9,200 tons of steel, 135,000 sq. yd. of pavement and 100,000 lin. ft. of piling.

At the start of the work the estimated cost was in the neighborhood of $24,000,000, of which $14,000,000 was for land and $8,727,000 for construction. At the present time the estimated cost of construction is $8,000,000. The construction contracts let total $6,751,026.42 and the work completed amounts to $4,714,104.91. It is a credit to all concerned that the work will be completed on schedule and at a saving of about $500,000.

Design.—The principal structural feature of the South Water St. Improvement is a reinforced-concrete flat slab viaduct, 114 ft. wide and about three-fourths of a mile long, with columns spaced 32 ft. 6 in. on centers. The average thickness of the slab is 17 in. The column spacing was fixed at 32 ft. 6 in. in order to provide the necessary number of traffic lanes on the lower level and so as to avoid the slowing down of the traffic which is bound to follow where columns or other obstructions hem in the driver. On the lower level of the Wacker Drive Improvement the driver has the feeling of the "Open Road" both day and night. The viaduct is probably the largest structure of this type of construction, and is remarkable also on account of the many large irregular panels which were necessary at curves and bends in the lower level traffic lanes. The columns rest on concrete caissons or cylindrical piers generally 4 ft. 3 in. in diameter, which are carried to hardpan between 65 ft. and 85 ft. below city datum.

Where it was impossible to place the piers directly under the columns, owing to the present freight tunnels and the proposed passenger subways, at street intersections—State, Clark, Dearborn, LaSalle, Wells and Lake Sts., the column loads are carried into the cylindrical piers by means of large reinforced-concrete foundation girders, some of which are 5 ft. wide, 12 ft. deep and 70 ft. long.

Transverse expansion joints are spaced about 120 ft. apart, adjacent portions of the structure being completely separated above the top of the cylindrical piers and foundation girders. The upper level roadway slab is waterproofed by a three-ply membrane waterproofing.

The design specifications used were the standard city of Chicago specifications with such modifications as the magnitude and the unusual character of the work demanded. Most of the stresses were checked by the slope deflection method. The reinforcing steel used was deformed bars of

the intermediate grade, and the design was based on an allowable unit stress of 16,000 lb. per square inch in the steel. The allowable unit stresses in the concrete were: 40 lb. per square inch for shear, 80 lb. per square inch for bond and 750 lb. per square inch for bending, which are the usual stresses for so-called 2,000-lb. concrete.

However, from actual tests we know that the concrete is good for 3,400 lb. per square inch and that the steel is good for 22,000 lb. per square inch or more, and the actual strength of the structure, as built, is therefore considerably in excess of the strength required for the loads which it is to carry.

This excess strength amounts to 38 per cent which—since the dead load remains constant—means that the structure would carry twice the live load for which it is designed—or 48-ton trucks instead of 24-ton trucks—and still be within the usual factor of safety.

Concrete Specifications.—From an engineering standpoint the improvement is primarily a big, outdoor, reinforced-concrete job and, although many branches of engineering were involved, it was the concrete construction that made the greatest demand on the time and effort of our engineers. We were determined to build both for strength and permanence, and, in view of the many failures, or near failures that have occurred in recent years in stadiums and other similar structures, we realized from the beginning that we could not reach our objective without the closest co-operation between our office and field engineers and without never ceasing vigilance.

Our concrete specifications in general are based on the Joint Committee's standard specifications. The specifications call for carefully graded stone for coarse aggregate and, to begin with, we had the greatest difficulty in finding a suitable material in the Chicago market, due, it would seem, to the fact that engineers generally do not insist on the proper grading of the coarse aggregate although it is well recognized that this is an important strength factor.

Only one kind of cement was used in order to maintain uniformity. The specifications call for cement meeting with the A. S. T. M. specifications with the additional requirement that the strength at 28 days shall be at least 20 per cent greater than at 7 days.

The concrete mixture for the most part is 1 : 2 : 4 with 7 gal. of water per sack of cement, but the contractor was at liberty to increase the amount of sand as much as 30 per cent if such increase in the fine aggregate were balanced by an equal reduction in the coarse aggregate. The contractor also was permitted to increase the water content of any mix so long as the water-cement ratio remained constant.

The use of the inundator was specified, the purpose of which is to eliminate any uncertainty as to the moisture content of the sand. The time of mixing was specified as 1½ minutes from the time that the last material goes into the mixer to the time the first comes out.

The consistency of the concrete was governed by the requirement that the slump was to be between 3 and 4 inches, except in special cases.

The specifications contain very minute instructions as to the procedure to be followed when concreting in cold weather. Thus—for 1 : 2 : 4 concrete, the amount of cement per cu. yd. was to be increased from 6 to 9 sacks; the mixing water was to be reduced from 7 to 5.8 gal. and the time of mixing was to be increased from 1½ min. to 2 min. for all concrete to be placed during the months of December, January and February, a sliding scale being provided for concrete placed during October, November, March and April.

Provision was also made for heating the concrete materials and for preventing the fresh concrete from freezing when the temperature was below freezing.

The specifications provide that concrete may be conveyed to a hopper on the form work by chuting, but the final disposition of the concrete in the forms shall be from concrete carts.

One of the problems met at Lake and Market Sts. is the maintenance of street car and elevated traffic. We had to excavate and remove the foundations for the elevated columns at the same time shoring them up and maintaining the traffic on the structure across the bridge. The elevated columns for the new location extend about 3 ft. above the present roadway. The old columns are naturally in the way of traffic on both the upper and the lower levels, and they are quite a serious menace to traffic, but we have no alternative but to leave them there.

On the northwest corner of Lake and Market Sts. there is a large shaft extending up about as high as the ceiling, covered on the outside with Bedford stone. That is the exhaust duct for the ventilation system for the lower level. The machinery house is on the lower level at the northwest corner of Lake and Market Sts., and extends from half a block east of Franklin St. to and including the ramp down south of Lake St. The machinery will provide a complete change of air every seven minutes. The ventilation duct from Franklin and Lake Sts. is below the lower level roadway. Small concrete pilasters, with metal tops, are provided back of the columns adjacent to the building for exhaust air to go in. The ventilation ducts are in all cases on the side opposite the river front. This is done in order to allow the fresh air to come from the river underneath the arches and through the structure, carrying out the foul air through the sand and then to be shot up into the atmosphere, because carbon monoxide has almost exactly the same weight as nitrogen and does not go up or down unless some form of propulsion is used.

The Bedford stone between Lake and Franklin Sts. is nearly all in position. The ramp on the river front connecting the lower level of Franklin St. with the upper level is faced with Bedford stone, and underneath the south end of the ramp there is a small area that has about 10 to 12 ft. headroom. This was made double decked to provide access from the part on the east side of the street to the dock.

The columns are all octagonal in form, 34 in. outside diameter. There are a few of them that are 42 in., octagonal in form. At the top of the

column are two small moldings and where the column capital joins the drop panel there is another invert molding. These moldings are put on because we pour the column shaft to the column cap and let that stand 24 hours and then pour the slab, when the slab is ready. Certain shrinkages occur and these moldings are placed there to cover these shrinkages. Hub guards are around the columns extending up 6 ft. 3 in. above the lower level roadway. They are placed at that height because an unloaded truck, due to spring action, is considerably higher than a loaded one, and we try to arrange the columns in such a way as to prevent the trucks from knocking off chunks of concrete on the octagonal corners.

On the lower level at Franklin St. we omitted a column from the traffic lanes and by an ingenious design of beams created what we consider a very fine traffic intersection. The beams in one direction are all parallel, while the other beams of necessity are placed at various angles.

The concrete at this point has been rubbed and will give a very good comparison of rubbed finish with the structure just east which has not been rubbed. At the northeast corner of Franklin St. is a small section of slab poured in December. This has received no treatment other than the removal of the form. The section just east of Franklin St. is typical of the South Water St. improvement. The improvement block by block is the same; you can take a block between Franklin and Wells and replace it between Wells and LaSalle, except that the blocks may vary a few feet in length. From State St. to Michigan Ave. it is all special.

At Wells St. intersection, a lot of difficulty was encountered, in that we had to maintain street car traffic as well as elevated railroad traffic. The abutments for the elevated are still on the lower level and will have to remain there until the elevated railroad company makes the changes in the present structure. At LaSalle St. intersection you will see a large girder that has been excavated about 19 ft. The top of the girder is 9 ft. below the ground. The bottom goes right through the old passenger tunnel at LaSalle St. The girder is placed across the existing street car tunnel; in LaSalle St. it goes past and misses one abandoned water pipe tunnel, one that carries a 36-in. water main and just misses the Illinois tunnel system on the south. There are altogether six tunnels at the LaSalle St. intersection.

On the east end, east of State St., we have run across a good many difficulties. On account of the insufficient records available it is almost impossible to make an accurate estimate of the character and amount of the different material encountered. Special conditions also would increase the cost of labor in removing material and preparing the site.

INSPECTION AND TESTS.

General.—The size of the job and the desire to maintain a rigid system of inspection and control have made it imperative that part of the field forces devote their entire time to the work of inspection and tests. This

work is carried out by a number of inspectors under the supervision and direction of the field testing engineer, who reports directly to the engineer in charge of field construction. A brief outline of the inspection and tests is as follows:

Cement Tests.—In general, cement bins having a capacity of 25,000 bbl. are assigned for storage purposes for this project. Samples of the cement are taken from the conveyor belt at the time a bin is being filled, two duplicate composite samples being taken for every 200 bbl. loaded into a bin. The loaded bin is sealed by an inspector of the Board of Local Improvements and no cement is withdrawn until all 7 and 28 day tests on the samples are available.

One set of the duplicate samples taken is tested at the laboratory of of the Board of Local Improvements and the other set by the testing division of the Department of Public Works.

Inspectors are stationed at the cement mill to break the seals on the bins, watch the loading of trucks, reseal the bins, and issue tickets of inspection to truck drivers. These inspection tickets are received by inspectors on the job and no cement is accepted unless accompanied by one of these tickets.

Aggregates.—The quantity available is investigated so that an adequate supply for the entire requirements of work ahead is assured in addition to the tests for impurities and sieve analysis. At the present date about eight different coarse aggregates and about five different fine aggregates have been offered for test and of these the ones best fitted for the work have been chosen.

Samples of aggregate are taken from time to time as delivered on the job for tests as to uniformity.

Proportioning Aggregates.—This is the work of combining the selected aggregates in the most advantageous way under the specifications to produce a strong dense concrete.

Samples of the selected aggregate for a given section of the work are sent to the laboratory for concrete tests. A sufficient number of batches are made to cover the range of proportions permissible under the specifications, and a 6 x 12-in. cylinder made from each batch.

Careful records of quantities of materials, slump, flow and yield of the batch are kept. The weight of the concrete cylinder is obtained immediately after the form is removed. Specimens are cured in damp sand until the date of the test and are tested at 7 and 28 days. All data taken from the batches are considered and the mix which gives a high strength and a good density selected; provided that it is workable.

Field Testing.—This work includes the sampling of freshly mixed concrete, the slump test for consistency, the making of 6 x 12-in. test specimens in a standard manner, the storing of the specimens and the test of the specimens under compression.

The compressive strength of the field test specimen has been the determining factor in deciding the date at which the contractor was to be

allowed to strip forms and have been given careful consideration when traffic was to be opened over a section shortly after that section had been built.

It is readily seen that a field test specimen which represents the actual condition of the work in the field is difficult to obtain. To core out a test specimen from the reinforced slab would have been impracticable. Therefore it was decided that as close an approximation to actual conditions as was possible should be obtained. From April to November, 7-day and 28-day specimens are cured in damp sand and duplicate 28-day specimens are stored 14 days in damp sand and 14 days in air. From November to March, specimens are placed on a platform under the slab in the heated area, and kept damp. Ten specimens are made from three different samples taken during an eight-hour period of concreting. Three are tested at seven days, two at fourteen days, one at twenty-one days, three at twenty-eight days, and one held for a test at a later date.

In addition to these, eight cylinders per eight-hour shift are made in paper molds, placed in a 2 x 4-ft. box on the upper surface of the freshly poured slab. As soon as they are made, they are completely surrounded with fresh concrete. The box is covered with hay and protected in a manner similar to the slab. Specimens are easily removed from the box on the date of test by breaking away surrounding concrete with a bull-point and sledge. Thus eight specimens cured on the upper surface of the slab are available for test from each eight hour period, two cylinders each for seven, fourteen, twenty-one and twenty-eight days, respectively.

Compression tests of the nine specimens cured under good conditions below the slab give values which are believed to represent the best condition of the concrete in the field. Tests of the eight cylinders in the box on the surface of the slab give values which are believed to represent the worst conditions.

Test specimens representing concrete in caissons, foundation girders, or other work placed below the ground level, are cured in damp sand in the field laboratory.

Whenever possible, test cylinders are capped with neat cement after the initial set of the concrete. If not capped with neat cement, specimens are capped with plaster of paris before testing.

Field Inspection and Control.—This consists of enforcing all the rules that lead to the best possible results, such as: water control, accurate measurement of materials, proper time of mixing, proper placing of reinforcement and concrete and proper curing conditions.

The stationary mixers on the job are equipped with a batcher for measuring the stone and an inundator for measuring the sand. These devices are carefully calibrated and set to deliver the proper proportion of aggregates as pre-determined by test. Where concrete is mixed in a portable mixer not equipped with batch measuring devices, two sample wheelbarrows, properly measured, one for sand and one for stone, are placed near the mixer in full view of the inspector and the men loading wheelbarrows.

In the latter case, the quantity of sand is increased to take care of bulking, the amount of increase being based on laboratory tests.

In placing substructure concrete, two inspectors are assigned to each mixer. One man remains at or near the mixer and receives cement inspection tickets, inspects the aggregates, checks the quantities of materials entering the mixer for each batch and is responsible for the proper time of mixing. The other inspector sees that the structure conforms to the drawings, inspects the quantity and placing of the steel, sees that the forms and reinforcing steel are clean, that the temperature and consistency of the concrete are acceptable, and supervises the placing of the concrete.

In placing superstructure concrete in the slab, the work is so arranged that concrete is placed in a continuous operation from expansion joint to expansion joint. This means a 36 to 96-hour period of pouring and necessitates dividing the inspection forces into three eight-hour shifts of six men each. One man is in charge of the inspection crew and his duties are to supervise the work of the men under him, see that the measuring devices are properly set, watch the temperature and consistency of the concrete and see that the forms are clean. One man inspects the placing of the reinforcement and two men inspect the placing and puddling or spading of the concrete. One man is stationed at the mixer to see that each batch is mixed a full minute and a half and that the proper amount of cement is placed in each batch. One man takes samples of concrete, makes slump tests for consistency and molds concrete test specimens.

Particular attention is paid to temperature of the concrete during the months of November to March inclusive. Each concrete inspector has a good grade thermometer, enclosed in a metal case, which can be carried in the pocket like a fountain pen. A thermometer is placed in the fresh concrete at intervals to make sure that the temperature of the concrete as delivered from the mixer is of the proper temperature. A record of the concrete in place in the work is obtained by frequent readings of a maximum and minimum thermometer or by a recording thermometer.

It is the duty of the inspector in charge, or the field testing engineer, to see that the concrete is properly protected against drying out and that it is kept moist from seven to fourteen days depending on weather conditions. In warm weather the surface of the viaduct slab or other horizontal surface is covered with about an inch of sand and kept moist for about 14 days. Vertical surfaces are sprinkled twice a day or oftener.

In winter weather, when salamander heat is applied, the surface of the concrete is covered with about a foot of hay which is kept moist and the under side of the forms is sprinkled occasionally.

Structural and Reinforcing Steel.—All metals required for the improvement, such as structural steel and reinforcing bars, are inspected at the place of manufacture by a representative of the Board of Local Improvements.

Samples of reinforcing bars are obtained from time to time on the job by cutting short lengths from bars selected at random from stock piles and check tension and bending tests are made.

Test Data.—The rigid system of inspection and control enforced on this project has been largely responsible for the fact that the concrete placed on this improvement has been of the highest quality.

The average compressive strength of 1:6 mix concrete, specimens stored in damp sand between May 1 and Nov. 1, is 1,760 lb. per square inch for 7 days and 3,300 lb. per square inch for 28 days. The average compressive strength for 1:5.14 mix concrete, specimens stored under conditions approaching concrete in place on job between Oct. 1 and Feb. 1, is 1,690 lb. per square inch for 7 days and 2,660 lb. per square inch for 28 days.

References.—For a more detailed description of the improvement reference may be made to articles which have appeared in the technical press as follows:

W. S. E. Journal March, 1925.
Engineering News-Record Oct. 15 and 22, 1925.
Concrete Sept., 1925.

Contractors.—The greater portion of the work has been let to the Mid-Continent Construction Company, but important parts of the work have been carried out by the M. E. White Company, The Underground Construction Company and the Great Lakes Dredge & Dock Company, all of Chicago.

The Engineering Staff.—The plans were prepared and the work is being carried out under the supervision of T. A. Evans, Engineer of Design of the Bureau of Design with the following engineers as principal assistants:

A. Engh, Assistant Engineer of Design, in general charge of office work.
G. F. Patrick, in general charge of field work.
R. O. Benson, in charge of drafting room.
G. Jeppesen, in charge of specifications and contracts.
G. A. Brown, in charge of utilities.
E. A. Howes and O. H. Taylor, in charge of accounts.
A. R. Lord is consulting engineer on concrete construction.
C. D. Hill is engineer of the board.

Ornamental Concrete Floor Surfacings with Especial Reference to Terrazzo.

By H. S. Wright.*

While the literature of concrete contains innumerable data on the subject of concrete floor surfacings, material having to do with the more decorative types is especially noteworthy because of its comparative rarity. A quarter of a century ago when ornamental concrete floors were just beginning to gain a foothold in the United States such a condition might have been attributed to the natural conservatism of technical writers. Today, when concrete floors of one type or another are the rule rather than the exception in all manner of buildings, there is no valid reason why they should be any longer neglected in print.

It is the writer's hope that the following skeleton notes on the subject of ornamental concrete floor surfaces will lead to other and more specialized papers on the individual varieties and in due course of time to the preparation of standards which will help to correct the errors of omission and commission that still occur in the building of these surfaces.

The term decorative concrete floor surfacings is somewhat ambiguous without specific qualification. In this paper such surfaces will be considered to include only the varieties of concrete floor surface which are commonly used in non-industrial types of buildings. A single exception will be found in a description of the processes of construction of terrazzo floors in industrial plants since the methods used are independent of the occupancy and may be applied to the construction of terrazzo floors in any class of building. The four classes of surface which qualify under these restrictions are the unit tile, granolithic with integral color, terrazzo and surfaces stained with chemical penetrants. Each of these general classes constitutes a separate group of similar surfaces in which the processes of construction are essentially identical although the constituent materials and refinements of manipulation may vary over a wide range. Except for the tile surfaces, all may be built either monolithically with the base or as a separate topping course laid after the supporting slab has hardened. All of them, in order to achieve a maximum of successful performance, require an intelligent selection of aggregates and admixtures, a firm unyielding support and meticulous adherence to the principles which govern the making of good concrete, especially curing. These principles are well covered in existing Institute floor standards and need no repetition here.

Tile Surfaces.—Tile surfaces are probably the ancestors of all modern concrete floors. Their original fabrication in concrete is somewhat obscure but it is reasonably certain that they were developed in Italy during the twelfth or thirteenth century as a means of overcoming the prevalent defects

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* Engineer, Structural Bureau, Portland Cement Association.

of brittleness and lack of endurance in the earlier forms of ceramic surfaces. Today many of our concrete floor tiles are made by Spanish and Italian artizans whose skill and knowledge of processes is a heritage from uncounted generations of tile making ancestors. The individual tile are still molded in forms by hand, subjected to a hydraulic pressure of many tons, laid aside in racks for moist curing and finished on a rubbing bed or with a hand hone. When properly made such tile are extremely dense, hard and durable.

Tile Varieties.—Two varieties of concrete tile are in common use in this country. These are the so-called Spanish tile and the terrazzo-mosaic tile, sometimes known as art marble. Spanish tile consist of a rather lean backing cast monolithically with a wearing surface of cement, coloring matter and sometimes sand. The terrazzo-mosaic tile are essentially a high-grade terrazzo made under pressure. Tile are made in a variety of sizes and shapes, ranging from 4- to 12-in. squares, hexagons, rectangular pieces or half tile and diagonal halves. In the Spanish variety a profusion of color is used either in solid area, unit design or parts of design which require several tile to complete any particular pattern. Variations in the type, color and size of the aggregate used in the terrazzo tile provide a wide range of varying effects and a somewhat more limited possibility of pattern design.

Abrasives such as carborundum or alundum are sometimes used in both varieties—to provide greater security of footing although the natural tile surface of cement and aggregate or cement and colors is by no means slippery. Of the two varieties terrazzo mosaic tile seem to have the greater present popularity in this country although in Europe and Spanish America the Spanish tile predominates.

Some very beautiful and enduring floors of the latter tile are to be found in Mexico City in bank buildings, libraries, churches and similar structures. Terrazzo or art marble tile have shown a consistently good performance in many of the newer railroad stations, libraries, museums and office buildings in this country. A notable example selected at random from a large list of satisfactory installations is the Madison Street level of the Chicago & Northwestern R. R. station in Chicago.

Method of Laying Tile.—Concrete tile may be laid over timber or concrete foundations with success, the more rigid base being preferable since it practically eliminates the tendency toward cracking through the tile surface which is especially noticeable when the tile are laid with broken joints. The customary procedure in setting tile is to lay a 2- to 2½-in. base course of lean concrete or mortar of about 1 : 5 proportion over the foundation course. This base should be bonded to the foundation if possible and struck off to a level at least the thickness of the tile plus ½-in. below the finished floor grade. A stiff brooming should be applied to the base just before the concrete is set in order to provide a better bond for the bedding mortar which is applied after the base has hardened but preferably within twenty-four hours. Before the tile are laid the base is

thoroughly washed by means of a strong hose stream and excess water removed leaving the base uniformly damp but not saturated. Tile are soaked for at least ten minutes in clean water and permitted to drain until the water just disappears from the surface. They should then be laid as rapidly as is consistent with good workmanship. Mortar used for setting or bedding the tile may consist of 1½ parts of portland cement, ½ part of lime and 5 parts of clean, sharp well-graded sand. Excessively wet mortar is disastrous since it results in water pockets under the tile and makes the work of laying to true line and grade exceedingly difficult. Only sufficient mortar for thirty minutes' use should be mixed at one batch and uniformity of mix and consistency is highly desirable. Strings or guides are stretched across the floor at exact right angles to each other and tile are laid in a mortar bed starting from the middle of the room or area to be covered. Where tile are laid in a warm atmosphere only sufficient mortar for eight or ten tile should be spread in advance of the actual laying. The width of joint used will depend to some extent upon the presence or absence of heavy wear. In places where wear is severe it is best to lay a hair line joint since any filler material commonly used is softer than the tile surface and when worn down by traffic may result in chipping or fracture starting at the exposed corners or edges.

After the tile are in place it is customary to permit them to set for several hours or until the operation of joint filling will not displace tile. The joint filler usually consists of a neat cement grout of creamy consistency which is brushed into the joints by means of a squeegee or soft broom. Workmen stand on boards laid over the tile as an additional precaution against displacement. Surplus grout is immediately rubbed off the floor using burlap or excelsior as a mop. After the grout has hardened any remaining traces or stains may be removed by scrubbing. In certain forms of terrazzo tile great care is necessary in removing the surplus grout so that it will not bond with the aggregates on the surface. Concrete tile floors are the equal in every respect of the better grade of ceramic products.

Integrally Colored Surfaces.—Granolithic surfaces containing integral coloring matter are as a group the least expensive of the decorative floor surfaces. Two main practices of laying prevail. In the first the coloring mixed with cement or sand or both is dusted on the floor after the topping material and laid as in a two-course job. In the second the color either mixed with cement or sand or both is dusted on the finer after the topping is spread and is then given the customary heavy troweling. Occasionally in certain of the proprietary color compounds used for this purpose other chemical substances are added at the time of manufacture with the intention of hastening the hardening, increasing the early strengths, hardening the finished surface or providing against the more common of the ills that usually result from a dusted on surface. Both of the foregoing integral color processes are subject to a number of shortcomings, the principal one being the difficulty of incorporating a sufficient amount of evenly distributed

coloring matter to produce a rich, permanent color without seriously impairing the strength or wearing qualities of the floor surface. Some coloring materials which contain finely-divided metallic iron may eventually show rust streaks or spots especially when the floor is subjected to a proper amount of curing under damp sand or sawdust.

A third method of obtaining an integral floor coloring has been developed recently. This process or method consists of a cement dye or color which is soluble in water and which may or may not involve a chemical

FIG. 1.—SATURATED CONDITION OF TERRAZZO FLOOR SURFACE DURING GRINDING OPERATIONS.

reaction with the cement. In this process a predetermined amount of dry color is added to the mixing water and permitted to dissolve completely before the aggregate and cement are added. This process is so new that we do not have adequate data on the durability of floors in which it is used nor the effect that such coloring media have on the strength of the resulting concrete. For light service, however, such floors present interesting possibilities. Before using any integral coloring material it is the part of wisdom to ascertain by actual test the effect of such colors on the strength of the concrete and behavior in contact with strong scrubbing materials. Most acids and many alkaline substances are enemies of integral colors. To guard against damage it is sometimes necessary to

treat integrally colored floors at frequent intervals, with wax or some other repellant or coating.

Terrazzo Types.—No single type of ornamental concrete surface has so many good points to recommend its use as the so-called terrazzo-mosaic floor. The very processes of its construction tend to eliminate many of the ills to which other types of surface may fall heir, either through abuse or a broader latitude of manipulation. Color ranges and textural pattern design possibilities are practically limitless in this type of floor while its adaptability to all varieties of use from heavy industrial occupancy to the most elaborate of ornamental buildings is unsurpassed. By a proper manipulation of the kind and size of aggregates and materials and variations in the finishing processes terrazzo floors may be made highly wear resistant or just adequate for ordinary needs, they may be slippery enough to meet the most exacting requirements for dancing or of the highest type of non-slip surfaces. The cost may be made to vary over a considerable range without sacrificing the essential requirements of service, and appearance, while except for such surfaces as contain highly processed forms of aggregates no patent royalty payments are involved in their construction.

Terrazzo, Construction Methods.—Two general methods or techniques of laying terrazzo floors are recommended. The first utilizes the principle of an integral or after-bonded top course while in the second type, the terrazzo work is completely separated from the foundation course, or structural slab. The first method without question is the more desirable from the angle of dead-load, resistance to impact and economy, but it has its limitations. It should be used only over a foundation slab of such rigidity and strength that temperature changes or excessive deflection will not result in surface cracking in the terrazzo. This type of surface is successfully used on floor slabs laid directly on the soil where adequate provision is made for sub-drainage or against unequal heaving due to frost and where contraction cracks have been guarded against by joints of proper design. It is also a satisfactory type over flat-slab or heavy beam-and-slab construction in which sufficient negative reinforcement is used. The integral type permits the designer to utilize the topping course as a part of the effective slab depth in figuring design requirements, but it is extremely difficult to build to a true level and usually delays other structural operations to some extent.

The after-bonded top eliminates difficulties due to delay but presents the much discussed problem of good bonding practice. It also is much easier to build to true level and to protect from damage caused by subsequent operations or weather. The after-bonded top introduces an extra dead-load of from 10 to 18 lb. per sq. ft. in the design of the structure, depending upon the top thickness.

Two notable examples of after-bonded terrazzo occur in industrial plants which have been built within the past five years. One is located in the plant of the Hilliard and Merrill Co. at Swampscott, Mass., while

the other is in the Woodward-Tiernan plant, Tower Grove Ave., St. Louis. The former floors, which cover approximately 210,000 sq. ft. of heavy-duty factory space, were built after a carefully conducted series of wear resistance tests had shown the superior wearing qualities of the terrazzo type over other surfaces. A description of these floors occurs in a paper presented before the 1922 convention of the American Concrete Institute by William M. Bailey, whose firm constructed the building and floors. A

FIG. 2.—CONDITION OF TERRAZZO FLOOR SURFACE AFTER ROLLING AND TROWELLING. NOTE COMPARATIVE ROUGHNESS OF FINISH BEFORE GRINDING.

somewhat more detailed description occurs in the *Engineering News-Record* of April 12, 1923, p. 600.

Terrazzo. A Specific Job.—The recently completed Woodward-Tiernan floors were built in accordance with data obtained from a somewhat different series of accelerated wear tests conducted by the Fruin-Colnon Co., builder of the floors. The aggregates were supplied on a fineness modulus basis and tests of relative workability resulted in the adoption of a topping mix consisting of one volume of cement to 1.7 volumes of trap rock graded from chips which would all pass a ½-in. square mesh to a 98 per cent. retention on a 100-mesh sieve, the coarser particles predominating. A somewhat leaner mix proved to be entirely satisfactory but was discarded arbitrarily because of mixer capacity limitations.

At the start of operations the foundation slab was roughened by picking and washing but in the major portion of the more than 300,000 sq. ft. brooming alone before the foundation slab had hardened. The clean of surface covered it was found that a good bond could be obtained by stiff brooming alone before the foundation slab had hardened. The clean base slab was moistened and covered with a creamy grout just ahead of the laying of the topping. Grout was in all cases covered before it had stiffened. The topping material was mixed mechanically to a very dry consistency, spread and struck off to screeds consisting of ¾-in. angle irons

FIG. 3.—SETTING SCREEDS BEFORE APPLYING TOPPING MATERIAL. BASE SLAB READY FOR TERRAZZO FINISH.

which had been previously leveled up. A dry mix of one part cement to 1½ parts of the trap rock was spread evenly over the surface and the entire floor given a lateral and longitudinal rolling with a light and a heavy iron roller. Rolling was followed by a single troweling with no attempt made to eliminate trowel marks.

After seven days of moist curing the floors were given an initial grinding with rotary machines using No. 20 carborundum blocks and powdered emery, with a copious supply of water. Scum and wastage from the grinding was immediately removed from the floor by washing and all pinholes and imperfections thus exposed filled with a stiff paste of cement and

carborundum (1: 1 mix) rubbed in with a steel float and a rubbing operation of the grinding machines. Wastage was again removed. The floors were permitted to set for several days and were then given a third grinding which left the surface in a very smooth and dense condition.

In certain sections of the floors colored pebbles were added to the mix to produce a livelier color effect. The difference in shape of particle, however, necessitated a longer period of curing before the first grinding in order to avoid picking out of individual particles under the grinder. One hundred and ten thousand square feet of floor was covered with a 1-in. topping while the balance was ¾ in. in thickness. While no attempt was made to produce a highly decorative surface, wear resistance being the paramount issue, these floors present a very slightly appearance. Slightly more aggregate exposure might be desirable in a more ornamental surface.

Where terrazzo floors are used in buildings designed to carry light loads, the tendency has been to lay a topping course entirely separated from the foundation slab. Specifications prepared by several of the most successful terrazzo workers concur in the requirement that such floors be built in the following manner. On top of the foundation slab is placed a cushion of sand ¼ in. in depth. This layer is entirely covered by a single ply of roofing paper which acts as an effective dam against bond between the base and foundation courses. On the paper is spread a 2-in. layer of concrete or mortar mixed fairly dry. A concrete mix of one volume of cement, two volumes of sand and four volumes of gravel or stone graded up to ½ in. is preferable to the 1: 4 mortar sometimes used. This base course is screeded off to a uniform level ¾ in. below the finished floor grade and allowed to stiffen. A ¾-in. wearing course of a mix of one volume of cement to two volumes of well-graded stone chips is spread upon the base struck off level and given a heavy rolling followed by a single troweling. This topping material must be of extreme uniformity to obtain the most even texture. Grinding and finishing methods are similar to those described in the previous paragraph, except that where a polish is desired the floor may be given the third finishing with the grinding machines used as a buffer.

Terrazzo, Dividing Strips.—In the earlier examples of terrazzo floors, neglect to make provision for flexibility in certain types of floors was the cause of a great deal of unsightly cracking. To overcome the difficulty recourse was had to dividing the floor area into numerous small areas separated by metal strips. While these division strips do not completely eliminate random cracking they have proven to be so successful that they are now supplied commercially and used in all separate top jobs of the better class. These strips add about 20 per cent to the floor costs. The strips usually consist of 20-gage brass although copper and zinc are sometimes used. They are set in the base course completely dividing both the base and wearing surface along the desired lines or pattern borders. Strips should be provided at borders, mat sinkages, and around large pattern areas. They are especially desirable over beams and

at intervals of not more than 10 ft. both ways in large field areas. Being of a medium soft metal they readily grind down with the floor and are not dangerous to traffic.

Marble chips are the customary form of aggregate used in terrazzo work but other forms of mineral substances are entirely practical and permit of greater latitude in wear resistance and color effect. Combinations of different materials often result in pleasing and unusual effects. Alundum or carborundum in addition to varying the texture and color add appreciably to the security of footing. White cement is sometimes used instead of the darker variety to accentuate certain colors or to produce white surfaces, while mineral colors mixed with the cement in safe proportions further broaden the possibilities of terrazzo.

Stained Surfaces.—The field of good concrete floors of the stained variety is limited by the difficulty of obtaining staining materials which will successfully resist the attack of cleaning preparations, wear, the ordinary foreign substances to which floors fall heir and the action of the chemical salts in the concrete itself. A further limitation is the relative difficulty of getting such materials to penetrate to a sufficient depth in a well-made concrete surface. Floor paints do not come under this classification since they are merely a superficial application and do not become an integral part of the finished surface. Many types of oils have been used experimentally and practically mostly with indifferent and temporary success or entire failure. Such treatments have a very limited range of coloring value and many have proved actually injurious to the floor itself by causing the formation of destructive chemical compounds.

A very interesting staining process which gives promise of having a high degree of successful use has been evolved recently in this country. In this process advantage is taken of the fact that certain chemical compounds combine with the lime salts in concrete to form highly colored crystalline materials that do not readily break down under the action of cleaning materials or wear. Before applying these chemicals the finished floor surfaces is treated with hydrochloric acid to remove the surface skin of cement or any laitance that may be present after which the chemicals are applied in successive treatments by means of a chisel-edge paint brush. Varying color effects and shades are obtained by variation in the chemical itself and by stopping the process at different stages of chemical action. As many as seven applications are sometimes required in order to obtain certain color effects. This process may be applied to either old or new floors. A fundamental requirement however is that the surface shall not be stained by oils, grease, lime spots or other dyes and that no admixtures be present in the concrete. It is also desirable to apply the process only after a floor is at least thirty days old or until the hydration of the cement is virtually completed. The essential requirement for success in a treatment of this type is a good concrete floor to begin with.

Conclusion.—Ornamental concrete floor surfacings are a logical devel-

opment of a civilization in which economy of cost and the utmost in utility must be attained without the sacrifice of appearance and safety to human life. In order that concrete in this form shall continue to serve humanity in an ever-broadening field it is incumbent upon builders and designers of floors to give of their ideas and experiences for only in this way is it possible to achieve the best in practical results. It is largely through recognition of this community of interest that cement itself has become so dominant and useful a factor in American building practice.

Efficiency in the Supervision of the Construction of Concrete Road Surfacing.

By J. L. Harrison.*

The U. S. Bureau of Public Roads subjected a number of concrete road construction jobs to detailed analysis in order to ascertain what causes prevented consistent production at a rate theoretically possible, and to discover if these causes of delay were avoidable. The analysis embodied answers to the following questions: What is full production? What causes interfere with it? and, Are they avoidable? Stopwatch studies were made on all minute operations in materials handling and mixing and placing of concrete. As a result of these studies better synchronization of operations was possible, with a material increase in hourly and daily production, and the attainment of a better average. The whole story is told in this paper by Mr. Harrison, highway engineer of the U. S. Bureau of Public Roads.

The construction of concrete roads is, in all essential particulars, a manufacturing enterprise. As do those enterprises more commonly thought of under that heading, it involves the use of machinery and labor in the conversion of raw materials into a finished product. It is, then, a complicated operation covering a number of fields, and a comprehensive study of efficiency will naturally cover all of these. Thus, in the first place, there is the matter of efficiency in the field of management, that is, efficiency in the conduct of the business transactions which are involved in any paving enterprise. In this field are such matters as buying, selling, which here would be more commonly termed bidding, for in highway work sales commonly are made on this basis rather than under the practices that govern other sales transactions, hiring men, purchasing equipment, etc.

Another of the major fields that must be covered in a thorough analysis

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of construction efficiency is the equipment field. All of the many kinds of equipment which may be obtained to perform a specific class of work are not of equal merit. Some do better work than others. Some do the required work with less power than others or require, incident to their proper operation, less labor. One machine may have a high rate of depreciation while another has a relatively low rate. These are merely a few of the angles from which equipment must be studied and the results subjected to economic analysis before even a fair approach to full efficiency can be had here.

A third field is the labor field. It is not necessary to more than mention this, for so much has been written on the question of motion studies that many engineers have been turned away from work in the field of efficiency because of prejudices which have been aroused by the extravagant claims made as to how the application of such studies can increase the output of the laborer. In spite of the popular feeling of doubt as to the value of such studies the speaker is thoroughly convinced that they have value and that, within reasonable limits, they may be used to advantage. Perhaps other fields could be properly enumerated as part of the general subject of efficiency in construction work but as this would be merely to add to the list that have not yet been subjected to exhaustive analysis in connection with the study, which the Bureau of Public Roads is making, of the elements of cost in construction work, it may be well to turn to the one other conspicuous field in which efficiency is required—the field of superintendency.

It is a little difficult to define superintendency. Here clarity rather than conciseness is desirable, so it will be defined by description, and to that end, attention will be drawn to the general nature of a concrete paving operation. It is not, in reality, a single operation. Rather it is, in fact, a series of related operations. One gang prepares subgrade. Another gang sets forms. The trucks deliver material. The mixer mixes it. Another crew finishes it, etc. These operations must be kept in harmonious relation with each other. They must, in short, be synchronized. It is not possible to mix concrete today and finish it next week. The material for next week's run cannot be delivered at the mixer today. The process is, and must be, one of continuous balanced production in all departments. But making it this and keeping it this is one part of superintendency. However, synchronization is not the whole of superintendency. Another part is the maintenance of proper rates of production. The mixer (under a specification requiring a one-minute mix) should produce 48 batches an hour. Superintendency carries the responsibility for eliminating any human or mechanical inadequacy that prevents the attainment of this production. Finally, superintendency carries responsibility for the elimination of useless effort. Thus approached, superintendency may be viewed as responsibility for synchronized effort, for the attainment of proper output, and for the elimination of all effort not necessary in the attainment of proper output, and full efficiency in this field may be defined as the attainment

of the fullest production legally possible from the equipment available, without waste or useless effort.

It may be remarked in this connection that superintendency is, in its very nature, an engineering field. It involves the collection of scientifically accurate data, its thoughtful study and its careful analysis. It implies action on the basis of fact—fact as developed by carefully collected and carefully analyzed data. Of course, it implies a thorough knowledge of ordinary practices, but as an absolute essential to success it also requires the deep conviction that ordinary practices probably can be improved. It is at this point that the current situation first shows its weakness. Far too many men now occupying positions in the superintendency field have a deep conviction that "whatever is, is right!" Not trained in the art of analysis, not accustomed to collecting facts and to studying them in an impersonal fashion, their tendency is to learn practice as one learns the multiplication table—as something which once learned can always be depended on.

"You can't move dirt with a pencil," said an experienced contractor who acts as his own superintendent to one of the Bureau's representatives who was collecting data preliminary to an effort to speed up this job. "Perhaps not," replied this young engineer, "but where they don't use pencils, they don't move much dirt!" In this bantering conversation was expressed all the difference between superintendence as an art—a knowledge of practice—and as a technical science—the application of general principles after thoughtful study and careful calculation. And, it may be added, that within a month the boy and his pencil had increased production 50 per cent.

The difference between superintendence as an art and superintendence as a technical science could be endlessly illustrated but that is hardly necessary in this connection. The purpose in drawing attention to the fact that there is a difference has been first, to show why such low standards of efficiency are accepted without question—a condition due entirely to the tendency of men who are learning any art to feel that once the art is mastered there is nothing further to do; second, to point out that if progress toward higher efficiency is to be made this must be accomplished by obtaining general recognition of the fact that superintendency is a technical science; and third, that, therefore, the field must, as rapidly as possible, be manned by men with technical training—men who are capable of approaching the very real problems presented every day to those who are working in this field, with the same confidence in the reliability of the laws governing their proper solution as is felt by the bridge designer or the technician in the testing laboratory.

Parenthetically it may be here observed that to the thoughtful engineer there is another aspect of this situation worthy of his most careful consideration. It is a common feeling among engineers that the profession is overcrowded. Probably it is. But if it is, this is because enormous fields, some of them, as in the case of superintendency, offering oppor-

tunities far beyond those now offered in the more ordinary and badly crowded fields, are being almost wholly overlooked by the young men entering the profession. Our great universities in their attitude on this matter, if this can be correctly judged by the courses of study offered, are much like shoe factories which have not changed their styles for twenty-five years. More leadership in the development of new fields in which engineering training can be effectively used is needed, and the lack of it is responsible for the inadequate training of the men today occupying such fields, just as it is responsible for the inadequacy of the training of the great majority of the men now occupying the superintendency field in highway construction work.

It may seem that in referring to the superintendency field as today not effectively manned, injustice is being done. That this is not the case can, however, be determined readily enough by a thoughtful scrutiny of production figures from any region of any considerable size in which extensive concrete paving operations are being conducted. If the output of twenty or thirty mixers is examined it will be noted that few come anywhere near obtaining the production theoretically possible. Indeed it will be found that in a considerable percentage of cases production is little if any over half of what it should be. There will also be cases where production is a good deal less than half of what it should be. But production is one of the outstanding responsibilities of superintendency. If the examination is extended it will be found that there is no consistency in the number of men employed. For instance, a superintendent producing 350 ft. of concrete a day may be using more men on the preparation of subgrade or in setting forms than the superintendent who is laying 800 ft. of concrete a day. Other matters equally important could be brought out, but as they all point in the same direction this is hardly necessary. As a matter of fact it was such analyses as these that suggested that perhaps the matter might be further analyzed, for while it was apparent that superintendence was not securing the results which appeared possible—the facts enumerated above fell short of laying the blame squarely on superintendency, for there was in them nothing to show that poor results could be replaced by better. Therefore, the Bureau of Public Roads determined to subject a considerable number of concrete construction jobs to detailed analysis for the purpose of ascertaining what causes prevented consistent production at the rate theoretically possible and whether these causes were avoidable. It was felt that it would be measurably determined that superintendence is primarily responsible for low production, if it could be shown that the direct causes of low production can be readily corrected.

The analysis of this matter has covered three distinct phases: first, What is full production? second, What causes interfere with it, and how much do they interfere? and third, Are they avoidable? To answer the first and second of these questions, the operation of a considerable number of mixers at work in the field was subjected to stopwatch analysis. Men were sent into the field and kept on a project for from a few days to a

couple of months collecting readings on all phases of the work. When the data seemed to show that an operation could be performed within a certain time, though it might not have been performed consistently within that time on the jobs studied, an effort was made to locate a job where it was so performed and there to study conditions with particular care. Finally as a means of determining that the causes interfering with production are avoidable, a number of jobs were found where the contractor offered to co-operate in an effort to demonstrate that prevailing causes of underproduction are avoidable by making such changes in construction practices as the stopwatch studies showed to be necessary in order to secure full production.

The studies which were made to determine what is full production began with a study of the mixer cycle. These studies at once showed that from 9 to 10 seconds are required to raise the skip. Of this time, the skip is discharging into the drum during from about 1½ to 2 seconds. After the skip is in vertical position, some time is required before all of the material which it contained is emptied into the drum. Different types of mixers differ a good deal in the time required here. The nature of the materials also affects the discharge time. The faster mixers generally take from a little under 4 seconds to a little over 5 seconds. An allowance of 5 seconds for the lag in discharging the skip is appropriate for such mixers. The time required in mixing a batch (over the larger part of the region where these studies were conducted) is sixty seconds after all materials are in the drum. After the bell on the timer rings it takes a little time to set the levers and for the discharge mechanism to work. This generally amounts to about 2 seconds. It may amount to more but when it does it is apt not to be uniform. Time so used extends the time of mixing correspondingly. The discharge requires about 10 seconds if a mix of proper consistency is being had. Sloppy concrete discharges faster because it does not pile up in the bucket or in the chute. Discharging the drum should, of course, overlap raising the skip. There results, then, the following mixer cycle:

Raising the skip with simultaneous discharge of batch....		10 sec.
Lag in charge	5 sec.	
Mixing time	60 "	
Total	65 sec.	
Less lag in discharge	2 "	
Set Timer	63 sec..	63 sec.
Lag in discharge		2 "
Total mixing cycle		75 sec.

This is 48 batches an hour.

If, now, raising the skip is delayed until the drum is emptied, the cycle is extended to 85 seconds and the possible production per hour is reduced to

a fraction over 42 batches—a reduction of between 12 per cent and 13 per cent. Again, if, as an illustration, the operator still further delays matters by not only emptying the drum but also running out and emptying the bucket before he starts the skip the mixing cycle is extended to about 90 seconds and output is reduced to 40 batches an hour, a loss of output amounting to about 17 per cent. It may be interesting to note that as a matter of observation and record, very few jobs have been found on which the operation of the mixer is so well handled that the 75-second cycle is attained with even fair consistency. The Bureau's representatives found one such job this summer. Full attainment of this cycle is hard to obtain but an average cycle within about a second of the ideal is not difficult to obtain with a good mixer as has been fully demonstrated by the Bureau's engineers who have found a substantial improvement of the mixer cycle generally the easiest part of any effort toward "speeding up" a job.

The correct mixer cycle involves one or two matters about which there may be some question. In the first place, if raising the skip and discharging the drum are overlapped, charging the drum and discharging it will occur simultaneously during the last 2 seconds of the period required for these operations. As the standard drum speed is about 15 revolutions a minute—one revolution every 4 seconds—such material as runs into the drum during this period cannot, of course, be raised high enough and moved over far enough to appear in the discharge. If any does appear it is an indication of too high a drum speed and the contractor should correct the condition by reducing the drum speed to normal, for, under current specifications, he obtains no advantage whatever from operating at a high drum speed. Just why specifications are written in this way the writer has never been able to learn. Specifications commonly permit speeds of from 14 to 21 revolutions a minute. Mathematical analysis suggests that at the higher of these speeds the action of the materials in the drum can hardly be sufficiently different from what it is at 14 revolutions to justify any other conclusion than that at 21 revolutions it receives approximately 50 per cent more mixing than at 14 revolutions, these remarks applying to a 6-ft. drum. If, on the other hand, the rate of revolution is increased much beyond this point, mathematical calculation indicates that the centrifugal force is likely to play an important part in reducing the mixing effect. The natural deduction from this is that while, under current specifications—a one-minute mix at about 15 revolutions per minute—the 75-second cycle is not likely to be much reduced by any change in mixer design, it may be practical to reduce it to about 60 seconds by increasing the mixer speed to about 20 revolutions per minute. If it should be found by test that this gives the same amount of mixing which concrete is now receiving, no important change in the general design would be required while, of course, possible production would be increased to 60 batches an hour, which is about as high a rate of output as, on the basis of data now in hand, it seems possible to attain under field conditions because of the numerous correlated operations that it would be difficult to hold constantly within any shorter time limit.

The second point relative to the 75-second mixer cycle is that, as mixers are now designed, the discharge must be forced to keep it within the 10-second limit on which this cycle is based. The bucket on most mixers is too small to properly accommodate a dry batch. Most mixers also dribble the discharge a little. Some dribble it a good deal. This is a matter governed by the design of the blades and the shape of the drum. Obviously, with all the batch in the mixer, the blades work through a "pool" of concrete lying at the bottom of the drum. As the discharge proceeds the depth of this pool decreases. If, then, the blade construction is such that more or less of the concrete which is raised for discharge falls clear of the discharge chute, the process can continue almost indefinitely. With most mixers, the dribble can be avoided by reducing the discharge time to 10 seconds which, during the discharge of the first few batches, will result in leaving a little concrete in the drum. Within a short time, the amount thus accumulated in the drum will be sufficient to force the discharge of a full batch during the 10-second discharge period. The amount thus left in the drum is not large and as it is, with full production, combined with a new batch every 1¼ minutes, no bad results can reasonably be ascribed to the practice.

It has been noted that one of the requirements in efficient superintendence is synchronization of all related activities. This, for such a manufacturing process as laying concrete pavement, suggests two things: first, that the mechanical unit which turns out the product (in this case the mixer) shall, if possible, be the pace-maker for the work, and, second, that the rate of production in all related or subordinate operations shall be governed by the pace-maker.

On a concrete paving job there are a number of operations definitely subordinate to the mixing operation. Materials must be loaded onto the transportation units. These are sand, coarse aggregate and cement. These transportation units, if they are trucks, must be turned around at the site of the work, backed to the mixer and their contents dumped into the skip. If, then, any one of these operations takes longer than is required to mix a batch of concrete, this becomes the pace-maker for the job. Thus, in the case of a poorly designed gravel loading bin, it sometimes happens that the time taken to load a truck is greater than the time taken to mix the batch. One such installation was found this summer where the time required in loading a single batch of aggregate at times averaged three minutes. The capacity of the mixer is of no consequence while such equipment is in use for it very effectively controls the rate of output. In this case, production was, of course, cut to less than 20 batches an hour and it is needless to remark that if such a device had cost nothing at all and had required no labor in its operation it still would have been an expensive installation for the extra time taken by the trucks in obtaining a load, not to mention the low efficiency at which the crew about the mixer had to work, were items of expense that far outweighed any saving obtainable from the material handling equipment.

Handling cement is more often the pace-maker on the job. This is outstandingly the crudest operation on most concrete paving jobs. It is, today, almost exclusively a manual operation. Generally some five or six men are employed in handling cement—about twice the number required for handling some five or six times the weight of coarse aggregate. The ordinary practice is either to throw the proper number of bags of cement into the trucks on top of the aggregate at the cement house, putting a man onto the truck at the mixer to empty them just before the batch is dumped or to send the cement to the work in separate trucks, placing it in piles along the road to be later emptied into the skip. In either event, the operation is apt to conflict with fast production. If the cement is sent out with the aggregate on single batch trucks, one man is all that can work effectively in unloading it. Sometimes one cuts the wire fasteners and another follows emptying the sacks. But emptying the cement is heavy work and if it is not carefully supervised the material supply will be held up a few seconds here and a few there to the material detriment of the day's run.

If the practice of sending out the cement in separate trucks and piling it along the road is followed, delays are still more frequent. The mixer cycle is 75 seconds. Of this time at least 17 seconds—10 to raise the skip, 5 to fully discharge it and 2 to drop it—are needed in handling the skip. More often 20 are taken by the mixer operator. Commonly, dumping the aggregate will take at least 15 seconds. There remain, then, with everything running smoothly and on schedule, not much more than 40 seconds in which to empty five sacks of cement. This is enough if two strong men are employed, but whenever there is any delay in the arrival of a truck, as inspectors commonly refuse to permit the cement to be dumped before the aggregate is dumped, dumping the cement causes an extension of the mixing cycle.

An appropriate remedy for this situation is the installation of some sort of practical mechanical means for elevating the cement at the cement house to a point from which it can be dumped into the batch trucks with the aggregate. It takes cement some time to harden, even after it is wet, so it could not be damaged by such a practice during the time it is in transit to the mixer. As a matter of fact, there is doubt in the speaker's mind as to whether, even after cement sent out as part of the batch, has stood for some hours, the most exacting laboratory technician could prove a measurable deterioration affecting any considerable proportion of it, if he would adopt the practical method of basing his conclusions on comparative samples of concrete mixed in the field. In any event, there is no probability that the practice of sending cement to the mixer as part of the batch with the addition of extra cement to any batches which inspection showed to have been visibly damaged by standing over night, could be responsible for any such variation in the strength of the concrete, as now results from ordinary variations in water content that pass utterly without note or comment on many construction jobs.

To just what absurd lengths the vagaries of inspection go, may be illustrated by a case reported to the writer by one of his assistants. The contractor was using four-batch trucks and was sending out the cement in sacks thrown onto the aggregate. Under the rules governing on this job he was permitted to have two trucks at the mixer with all cement emptied. One evening a breakdown at the mixer prevented the use of two such truck-loads (8 batches in all), so the contractor covered them with a heavy tarpaulin and left them for use in the morning. There was no rain that night. The inspector's first act the next morning was to condemn these batches and personally to conduct the trucks to a high fill where the contents were dumped in such a way as to make any recovery impossible. The contractor offered to put an extra bag of cement in each batch. He asked to be allowed to take off the obviously good cement on the top and use that. There was no compromise. It all went over the bank! There is too much inspection of this sort in all phases of concrete pavement construction, but it is perhaps a little more arbitrary and a little more irrational where the cement is concerned than at other points with the result that contractors seem not to have dared here to work toward the modern methods that are in use in other parts of this work.

Turning the trucks and backing them to the mixer does not take time enough so that this operation becomes the pace-maker on the job, but cases have been observed where dumping the trucks has had the effect of controlling the rate of output. Dump truck bodies are of a good many styles. The desirable bodies are of steel and are so designed that while dumping a high angle is maintained, this resulting in a rapid flow of materials. With everything running smoothly, dumping can be done in a very short interval of time but, if the trucks must be dumped by hand, as is common with single batch trucks, a poorly placed load will sometimes result in considerable delay. However, it is only when home-made wooden dump bodies are used that the delay is at all likely to be consistently serious. On one job studied this summer there were a number of trucks equipped with wooden dump bodies which quite consistently used over a minute in dumping. As the period (see above) between the moment at which the skip is lowered, and at which it starts up again with another load, is generally less than 55 seconds, their inability to dump a load within a minute, made these trucks the pace setters to the extent of the proportion of the material which they delivered.

The other aspect of the production problem as referred to above, the synchronization of all activities, covers such matters as the amount of forms set, the rate of material delivery, the rate of finishing and curing, etc. It is not necessary to treat these at length for it is, of course, obvious that material can be mixed only as it is delivered and that, therefore, the rate of delivery must be based on the rate at which the mixer is expected to operate. In the same way, finishing must keep pace with production and the crew must be selected and trained with this in mind. Perhaps it is not quite as clear that form setting, and finishing the subgrade, are in the

same category, but a little thought on this matter will serve to convince almost anyone that this is the case. There certainly is nothing to be gained by setting forms twice as fast as concrete is poured—and indeed this is not possible for the simple reason that no job has enough forms to last long at any such rate. And as for finishing the subgrade, there is too much risk

TABLE I.—STOPWATCH STUDIES ON CONCRETE ROAD CONSTRUCTION.
Study No. 58-B.

Date, June 20, 1925.
No. of Batches Placed, 31. Time, 1 hour.

Mixer Cycle, time in seconds.			Time (in seconds) Lost by Mixer Due to the Following Causes.			
Charge.	Mix.	Discharge.	Mixer Delay Due to Operator.	Insufficient Truck Supply.	Trucks Delayed Because of Mechanical Trouble.	Delays Due to Miscellaneous Causes.
10	59	10	5	..	..	..
9	59	10	20	..	..	..
9	59	10	5	..	..	..
10	59	10	19	..	..	..
9	59	10	..	90	..	..
9	59	10	2	.	..	..
10	59	10	..	25	..	..
10	59	10	..	66	..	..
9	59	10	..	57	..	..
10	59	10	5	..	..	..
10	59	10	..	9	..	..
10	59	10	..	101	.	..
9	59	10	7	..	..	..
10	58	10	10	..	..	..
10	58	10	5	..	..	..
9	59	10	..	77	17	..
10	59	10	..	..	22	..
9	58	10	32	..	..	..
9	59	10	16	..	..	..
9	58	10	10	..	..	..
10	58	10	..	..	..	..
10	58	10	42	..	..	..
9	59	10	36	..	..	..
10	59	10	8	..	..	..
9	59	10	16	..	..	..
9	59	10	9	..	..	..
10	59	10	..	57	..	..
9	59	10	..	21	..	..
10	59	10	20	..	..	..
10	59	10	40	..	..	..
10	59	10	..	..	..	316
Averages...... 9½	59	10	10.2	16.2	0.1	10.2
Percentage of Working Time. 8	50½	8½	9	14	1	9

involved, too much chance that rains and traffic will destroy the work to justify keeping it far in advance of the mixer.

The second phase of the Bureau's studies in this field dealt with the matter of delays, their amount and their cause. To develop these, stopwatch studies have been made on a number of jobs. The general practice has been to take an hour's reading morning and afternoon, recording the

component parts of each mixer cycle and the length and cause of each delay. Such a record is shown in Table 1. These records were collected over such a period as seemed necessary in order to obtain a fair average of operation, the hourly studies being tabulated as in Table 2 for more general study. The tables used in illustration are perhaps not quite as typical of conditions generally as some which could have been shown but are given here because they show the conditions prevailing on one of the jobs on which an effort was subsequently made to improve production on the basis of the data which these studies developed. The graphs in Fig. 1 show the daily and the hourly production in lineal feet over a more extended

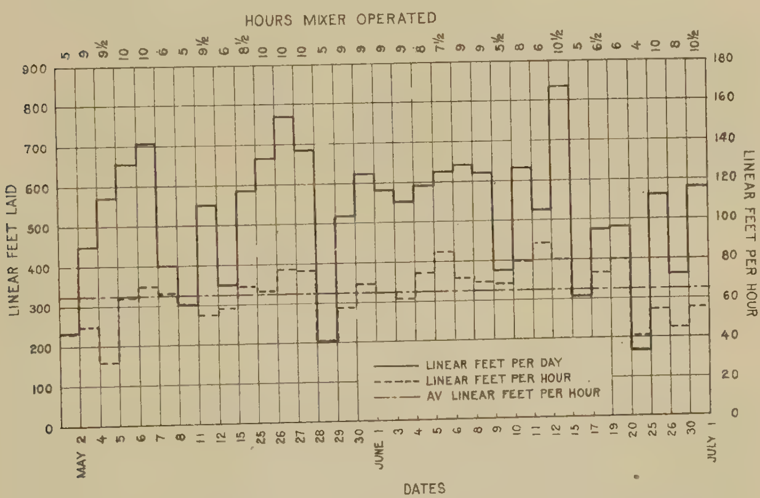

FIG. 1.—DAILY AND HOURLY PRODUCTION ON FEDERAL-AID PROJECT NO. 235, HOWARD CO., MISSOURI.

period. What these data show, responsibility for the conditions prevailing, and the steps taken for the correction of improper conditions will be discussed under the various headings used in these tables.

The summary of the studies set down in Table 2 develops the following facts:

Batches mixed per hour........................ 27.4

Per Cent

Average efficiency of production.................. 57.1

Average lost time.............................. 42.9

The lost time is accounted for as follows:

Actual mixer cycle............... 80.2 sec.

Correct mixer cycle.............. 75.0 “

	Per Cent of Working Day
Resulting loss	3.8
Mechanical trouble with mixer	4.9
Poor operation of mixer other than incorrect cycle	8.4
Shortage in truck supply	13.6
Time lost in handling trucks	4.0
Water supply	2.0
Trouble with subgrade	1.6
Trouble with finishing	4.0
Miscellaneous	0.6

Among these delays there are two outstanding groups—delays incident to the operation of the mixer and delays arising from the delivery of material—which here, as on practically all of the jobs so far studied, are the major causes of underproduction.

The delays incident to the operation of the mixer were, in this case:

	Per Cent of Working Day
Improper mixer cycle	3.8
Other improper operations of mixer	8.4
Total loss due to poor operation	11.2
Mechanical troubles	4.9
Total loss at mixer	16.1

The extension of the mixer cycle beyond the proper limit was due to the fact that the operator failed to overlap raising the skip and discharging the drum. On this particular job, the inspector had failed to set the timer to take care of the lag in discharging the skip, with the result that the contractor had here an advantage of perhaps 3 seconds over what should have been required of him. In spite of this advantage, his mixing cycle was approximately 5 seconds long. In addition to this, he had a habit of splitting batches and often ran out the bucket and dumped it before starting the skip. He was also in the habit of moving the mixer while it was empty. The accumulation of such erroneous practices as these accounts for the 8.4 per cent of the working day charged to poor operation, other than an improper cycle. It is hardly necessary to remark here that any really first class mixer has a power plant quite capable of handling the discharge mechanism and raising the skip while at the same time driving the drum under full load. Moreover, once the discharge has been started and the skip lifted off the ground, the power plant should be capable of moving the mixer while the discharge continues and the skip is being raised. There is, then, no power plant limitation in any first class mixer which prevents operation within the cycle outlined above, except when worn out mixers are being used. On the other hand, there are mixers which are function-

ally inadequate, due principally to poor blade design. One of these recently studied required approximately 30 seconds for emptying the skip after it had reached vertical position. A number of other machines have been found which commonly required a little over 15 seconds to accommplish this. It is, of course, quite unnecessary to observe that such mixers generate long mixer cycles and that, therefore, wherever there is functional inadequacy in the mixer itself, production at the standard rate of 48 batches an hour cannot be legally had.

TABLE II—STOPWATCH STUDIES ON CONCRETE ROAD CONSTRUCTION.

Date, 1925	Number of Batches Mixed per Hour.	Mixer Cycle, Time in Seconds.			Per Cent of Working Day Lost by Mixer from Following Causes.								
		Charge.	Mix.	Discharge.	Insufficient Truck Supply	Truck Delayed Because of Mechanical Trouble.	Mixer Trouble, Mechanical.	Mixer Delay Due to Operator.	Water Supply Trouble.	Preparing Sub-grade.	Lack of Materials Due to Poor Supervision.	Delay Due to Concrete Finishers.	Miscellaneous Delays.
6/11..........	16	9½	59½	10	..	..	..	15	..	..	..	50	..
6/12..........	36	9½	61	10	..	..	..	10½	..	9	..	..	..
6/12..........	31	9	60	12	..	9	..	16½	..	5	..	..	..
6/15..........	36	9½	61	10	..	9½	..	6	2	1½	..	..	..
6/15..........	34	9½	61½	10	..	5	..	13	6	..	..	..	..
6/19..........	31	9	58	12	..	3	..	17½	..	5½	..	7	..
6/20..........	38	10	59	10	10	1	..	6	1	..	..	..	..
6/20..........	38	10	59	10	6	..	..	8	2	1½	..	..	..
6/20..........	31	9½	59	10	14	1	..	9	..	..	..	..	9
6/25..........	10	10	61	10	13	21½	41½	1	..	..	..	..	..
6/26..........	17	9½	61	10	(1)28	1	27½	1½	4	..	..	..	..
6/26..........	24	9	60	12	(1)39½	..	..	4½	2	..	..	..	..
6/30..........	20	9½	61	10	(1)49	3	..	3	1	..	..	..	..
6/30..........	23	9	61	12	(1)30½	2	..	6	10	..	..	..	..
Averages......	27.4	9.5	60.1	10.6	13.6	4.0	4.9	8.4	2.0	1.6	..	4.0	0.6
Per cent of Possible Efficiency	57.1	80.2 seconds Mixer Cycle											

Mixer was idle 39.1 per cent of working day.
Additional time lost due to slow operation of mixer, 60.9-57.1, 3.8 per cent.
NOTE: (1) Very inadequate supply of trucks developed when hauling distance changed from one mile to five miles.

On the job under discussion no functional inadequacy existed. The improperly long mixer cycle and the other time losses incident to operation were entirely chargeable to the operator. The problem was, then, one of training the operator to handle his machine correctly. To this end, he was instructed:

First, never to split a batch.

Second, to empty the bucket only after the skip had been discharged.

Third, always to move his mixer while it was mixing.

Fourth, always to throw his skip lever and his discharge lever together and as quickly as he could after the bell on the timer sounded.

TABLE III.—STOPWATCH STUDIES IN CONCRETE ROAD CONSTRUCTION.
Study No. 58-B.

Date, July 31, 1925.
Number of Batches Placed, 47. Time, 1 hour.

Mixer Cycle, time in seconds.			Time (in seconds) Lost by Mixer Due to Following Causes.			
Charge.	Mix	Discharge.	Mixer Delay Due to Operator.	Insufficient Truck Supply.	Trucks Delayed Because of Mechanical Trouble.	Delays Due to Miscellaneous Causes.
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	3	..	..	..	..
10	64	3	..	..	..	..
10	64	3	..	..	..	..
10	64	1	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	63	2	..	..	..	..
10	63	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	63	2	..	..	..	..
10	64	3	..	..	..	..
10	64	2	..	..	..	..
10	64	2	..	..	..	..
10	63	2	..	..	..	..
10	64	2	..	..	..	..
Averages......10	64	2.5	..	..	..	..
Percentage of Working Time.13.5	83.5	3	..	..	..	..

Fifth, always to close the discharge so that it would reach full cut off position just as the skip reached full vertical position.

How completely it was possible for him to attain these results will be apparent from a study of Table 4 which gives the readings taken during the last week's operation on this project, during which time improper handling of the mixer—in this case too long a cycle—accounted for a loss of only 2 per cent of the working time. This is a good record for it must be remembered that any variation from the correct cycle must be an increase. It is not possible to average at 75 seconds for that could be attained only by shortening the time on some of the batches.

TABLE IV.—STOPWATCH STUDIES ON CONCRETE ROAD CONSTRUCTION.

Date, 1925.	Number of Batches Mixed per Hour.	Mixer Cycle, Time in Seconds.			Per Cent of Working Day Lost by Mixer from Following Causes.								
		Charge.	Mix.	Discharge	Insufficient Truck Supply.	Truck Delayed Because of Mechanical Trouble.	Mixer Trouble, Mechanical.	Mixer Delay Due to Operator.	Water Supply Trouble.	Preparing Sub-grade.	Lack of Materials Due to Poor Supervision.	Delay Due to Concrete Finishers.	Miscellaneous Delays
7/27..........	45	10	64	3	..	2½	1	..	1	..	..	..	..
7/28..........	44	10	64	2½	..	5	..	..	½	..	..	..	..
7/28..........	43	10	63½	2½	..	8½	..	..	..	..	..	..	½
7/28..........	44	10	64	2½	..	½	..	..	..	..	6½	..	..
7/29..........	45	10	63½	3	..	..	..	..	4½	..	..	..	..
7/29..........	46	10	64	2½	..	2½	..	..	..	..	..	..	..
7/30..........	38	10	64	3	..	..	..	..	2	4	13	..	..
7/30..........	38	10	64	3	..	3	..	..	16	..	..	..	..
7/31..........	47	10	64	2½	..	..	..	..	..	..	..	..	..
7/31..........	43	10	63½	3	..	2	..	..	1	5½	..	..	..
7/31..........	46	10	63½	2½	..	..	..	..	2	..	..	..	..
8/1..........	45	10	64	2½	..	..	..	..	4½	..	..	..	..
Average.......	43.7	10	64	2.7	..	2.0	..	..	2.6	0.8	1.6	..	..
Per cent of Possible Efficiency	91.0	76.7 seconds Mixer Cycle											

Time lost by mixer, 7 per cent of working day
Additional time lost due to slow operation of mixer, 93.0–91.0, 2 0 per cent of working day.

The conditions existing on this project and the change it was possible to effect through instruction of the operator in the proper manipulation of his machine justify two or three observations. The first of these is that mixer operators commonly have been found to have only a limited appreciation of what their machines should be capable of doing and of how to make their machines function correctly. To at least a certain degree this is due to the fact that manufacturers do not fully instruct their representatives as to the capacity of their machines and how to operate them. However, a larger factor is the tendency of contractors to assume that any good laboring man can be trained to run a mixer. Practically, it has been found that this is true but it has also been found to be even more com-

monly true that the training is not properly attended to. During the past summer the Bureau's representatives have dealt specifically with this problem on some seven or eight mixers working under widely different conditions and in no case have they failed to obtain definite improvement in operation by the relatively simple process of explaining to the operator how his machine should be handled and showing him the result of changes made in his manner of operating. This leads directly to the second observation, namely, that the unescapable deduction from these experiences is that responsibility for lost time from this cause rests squarely on the shoulders of the job superintendent. Existing conditions are within his obvious jurisdiction and the failure to secure proper results can be charged only to him.

The loss of time due to mechanical trouble with the mixer was a large item on this job. It has been found to be a large item on many jobs. Mixers wear out and like all other items of equipment must be replaced. They are, however, in a category somewhat different from the great bulk of equipment. With most of the equipment there is an opportunity to "catch up" any time lost in making minor repairs. In the event of a serious breakdown some other method of operation can often be arranged which will enable production to continue without the machine until it can be fixed. In other cases there may be a number of units employed—as, for instance, trucks—in which case a breakdown will affect only a fractional reduction in production. But with the mixer, any loss of time is instantly reflected in the production. Indeed, to stop the mixer is to stop production. This is wholly unavoidable. Therefore, the mixer should receive outstandingly good care and ought to be replaced more often than most other items of equipment. The payroll on a concrete paving job commonly runs in the neighborhood of $200 a day. Depreciation on equipment is at a rate generating a cost of perhaps half that amount, often more. These two accounts, to mention none of the accounts of lesser importance, can be said, in a general way, to amount to $300 a day or to 50 cents a minute. If, then, the mixer is losing 5 per cent of the time, this is 30 minutes, worth $15 a day, or somewhere between $1,500 and $2,000 during a good working season.

Mechanical difficulty may be due to the use of a worn-out mixer, to occasional breakdowns which, in turn, are likely to be the result of poor operation, or to careless or indifferent maintenance. If, from the latter causes, responsibility for them, as in the case of the poor performance of the operator, rests squarely on the superintendent. In the case in hand, the almost total absence of mixer trouble during the period covered by Table 4 is to be ascribed, at least in part, to the more complete training of the mixer operator and to the greater emphasis laid on proper care of the mixer. It was a comparatively new machine and should have given little trouble. If mixer trouble is due to the fact that the machine is worn out it can be corrected only by a complete overhauling or by the purchase of a new machine. To work with a machine that is losing upward of 5 per cent of the working time is too expensive to be considered for a moment by

a wide-awake contractor. This statement also applies directly to the use of functionally inadequate machines. For instance, a machine that, through poor design, generates a mixer cycle of 85 seconds throws away about 12 per cent of the working day (when worked to capacity), that is, about $35 a day or from $4,000 to $5,000 during a good working season. No contractor can afford to own such a mixer!

Another observation which may appropriately be made at this point is that the correction of conditions at the mixer, at once affects losses at other points, notably in the transportation supply.

In the case at hand, losses which had been amounting to over 16 per cent of the working day were, by careful instruction of the operator, (see Table 4) reduced to 2 per cent of the working day. As an inadequate truck supply already was responsible for a loss of 13.6 per cent of the working day, the corrections made at the mixer could have had no value whatever without a change here. They would, in short, merely have operated to increase truck shortage to 27.7 per cent. The objective in a construction operation is production. Obviously, the greater the production per man and per machine, the more favorable the position of the contractor. Any effort at efficiency which falls short of this result is a hopeless failure. But, in the studies of efficiency which have so far been made, there is one thing that has been conspicuous,—that it is not, on the whole, more effort per laborer employed which is needed. Rather it is more effective instruction of the men and more effective planning of their work. Laboring men at once sense any lack, either in knowledge of the work or in ability properly to direct it, but they respond at once to the efforts of any leadership which enables them to work more effectively. So, while it has often been observed that mixer operators, as a single example, are difficult to handle if an effort is made to correct conditions at the mixer, when ordinary practices there generate a rate of production which exceeds the possible rate of material delivery, it has been the uniform experience that, if crowded by the development of an improved rate of material delivery, they will work diligently to master an improved system of mixer operation. It is, therefore, not only useless so far as the effect on output is concerned to improve the operation of the mixer if the material supply, where already inadequate, cannot be improved, but it is also likely to be impossible to correct the rate at which other operations are performed, for the men involved are likely to so keenly appreciate the utter futility of it all that they will not respond.

On the job under discussion, the material delivery losses were:

	Per Cent of Working Day
Inadequate truck supply	13.6
Improper handling of trucks	4.0
Difficulties with water supply	2.0
Total	19.6

In making these studies readings were, of course, made at the mixer. Improper handling of trucks there, slow dumping and difficulty in handling caused by bad subgrade were causing a loss of 4 per cent of the working day. Slow dumping was later largely corrected by placing the stone in the back of the trucks which were of the single batch, hand dumped variety. This largely eliminated time losses due to slow dumping. The bad subgrade conditions were never fully eliminated. This tended to correct the deficiency in truck supply but could not, of course, eliminate it.

The fact of a truck shortage as shown by readings at the mixer indicates merely a shortage under existing operating practices. Single batch trucks travel at an average rate of about 15 miles an hour. The Bureau's studies show that about 4 minutes are required per load hauled, for servicing operations such as loading sand and gravel, loading cement, traversing loading plant, operation of turntable, etc. These facts generate the formula

$$T = 8d + 4$$

in which T is the trip time in minutes and d is the distance from the material loading plant to the mixer in miles. With this formula, it was possible to subject the existing truck supply to analysis to ascertain how many loads per day should have been delivered. By comparing the number of batches which should have been delivered with the number actually delivered, it was possible to determine the efficiency with which the trucks were being operated. Fig. 2 gives graphs showing the efficiency of truck operation under the contractor's management and as developed under the guidance of the Bureau's representative. For the period under discussion the average truck operating efficiency was about 64 per cent. In short, instead of an actual deficiency in the truck supply, the average truck supply available during this period was such that, had it been properly operated, a surplus would have resulted. This statement, of course, deals with averages and is satisfactory for a general analysis of the transportation situation, but for immediate job control, it is not sufficient, for averages, to be really significant, must deal with a situation in which there is opportunity to exceed a desired condition as well as to fall below it. In work of this kind, this cannot be done, for today's surplus truck supply cannot be used to increase today's run or to improve tomorrow's, if there will then be a deficiency. To correctly appraise the situation for proper job control, each day's work must, therefore, be considered by itself. Viewed from this angle, there would, at times during this period, have been some truck shortage, if all operation had been on a high plane of efficiency and no additional trucks had been made available. This will, perhaps, be a little clarified by the graphs in Fig. 2 which give some of the details as to the truck supply and its management.

The detailed analysis of efficiency with which the trucks were being operated included stopwatch analyses of service rendered the trucks, i. e., loading, turning, backing and dumping, etc., for the purpose of learning

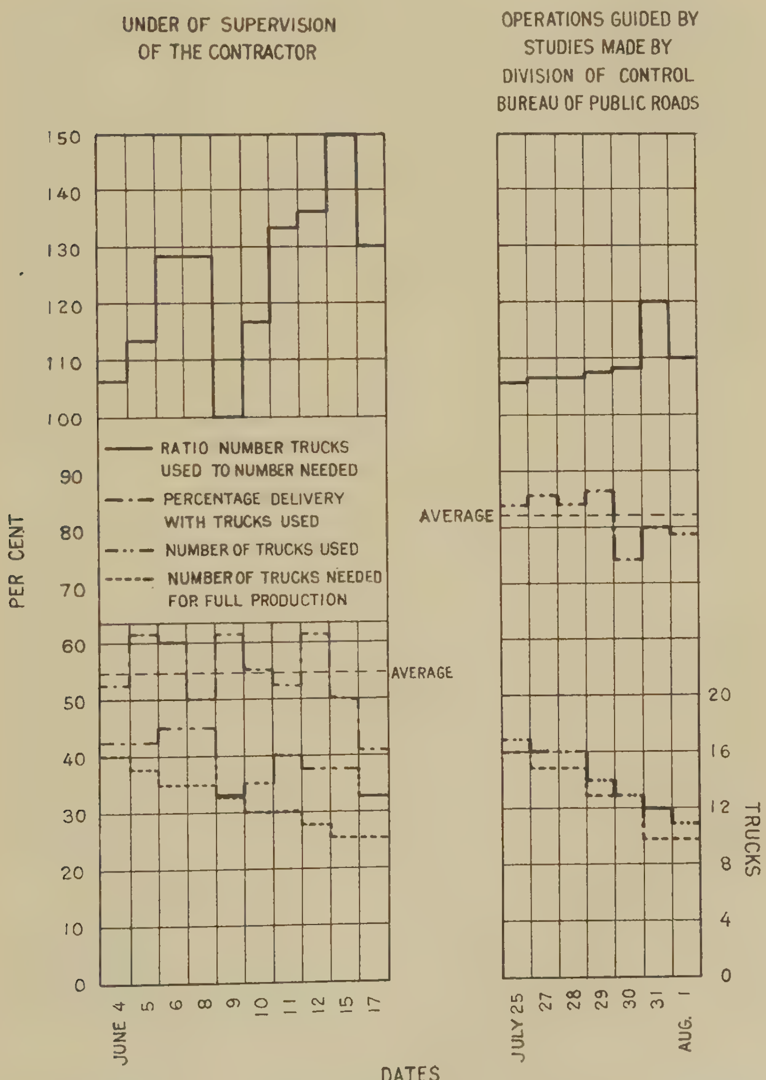

FIG. 2.—STUDY OF TRUCK OPERATION ON FEDERAL-AID PROJECT NO. 235, HOWARD CO., MISSOURI.

and correcting the various causes of delay. The stopwatch studies also developed the trucks which habitually ran either fast or slow and thus interfered with a uniform delivery on schedule time. The details of this work need not be discussed at length here but a few observations may be of interest. Thus, on work of this kind, it has been uniformly found that where a uniform travel speed it not maintained efficiency is reduced. If trucks are run at a speed greater than that established for the job, they are worn out more rapidly with increased depreciation and repair cost as well as a higher percentage of time lost in the repair shop. Accidents enroute, collisions, etc., are of more frequent occurrence. In short, general confusion in operation results, with nothing much on the other side of the ledger, for, though the truck which has been speeded up may obtain a few extra loads during a day's run, these are commonly obtained at the expense of some other truck so that no increase in amount hauled, taking the fleet as a whole, results. This arises from the fact that each servicing operation takes time and that, therefore, while the speeding truck may come in ahead of its place in the line, this merely results in its reaching the material yard with some other truck or trucks which must then wait for service, losing on the whole about as much time in going through the material yard as the speeding truck has gained.

The slow truck, of course, shows a direct loss of time itself, and also causes those delayed by its slow driving not only to lose time on the road but also to lose time waiting for service in the material yard. The net result, then, of both slow driving and fast driving is to injure the contractor, and whenever either is found to exist, if the drivers fail to coöperate in correcting the condition, they should be summarily discharged.

On the other hand, it may be remarked that while high production requires a rigidly scheduled delivery of material it is unfair to the men to assume that unassisted they can accomplish this result. Indeed it is as hard to maintain the material delivery schedule without adequate supervision as it would be to maintain a complicated train schedule without a dispatcher. Good supervision is essential. The men should be given proper information as to the trip time which is to be required. Speedometers would be a great assistance to them in maintaining the proper speed. They should be rigidly held to proper performance but they should be given adequate means of determining that they are rendering proper performance. Here again superintendence now is wholly inadequate. It is not enough that men should be told what is desired. The conditions surrounding their work should be such and their detailed instruction should be such that the result desired can be secured.

Another matter relative to the truck supply is that, in practice, even with the best of supervision, perfect performance is all but impossible of attainment. The elimination of delays in servicing and the introduction of a standard driving speed will improve material delivery. On another project, studied by another of the Bureau's men, the truck supply was inadequate throughout the period during which an effort was made to

improve production. Under the contractor's management the efficiency of truck operation had commonly ranged between 70 per cent and 75 per cent. By correcting the difficulties arising in connection with servicing, by keeping the trucks on schedule, etc., the efficiency of operation was raised to about 98 per cent, but it could not be raised to a full 100 per cent for any extended period and held there. A punctured tire, a train at the railroad crossing, a momentary holdup due to other traffic on the road, not to mention more of the numerous occasionally encountered causes of minor delay, would cause the trucks to fall a little below full efficiency. From this and other experiences with this problem, the conclusion has been reached that it is sound practice to use a truck supply somewhat in excess of that theoretically required. On the job under discussion, one extra truck was used, except that on one day two extra trucks were used, but the results obtained do not indicate that any increased rate of production resulted.

Where an excess truck supply is used the computed efficiency of operation will, of course, be correspondingly reduced. This explains the fact that an operating efficiency generally ranging between 80 per cent and 85 per cent was maintained during the high production period shown in Tables 3 and 4 and in Fig. 3. In the last analysis the question as to what excess truck supply it is well to maintain must be determined by the conditions prevailing on the job in hand. It has been shown that in a general way time lost at the mixer costs the contractor 50 cents a minute. The loss, for instance, of 5 per cent of the working day costs him perhaps $15. The loss of 10 per cent of the working day costs him perhaps $30 a day. The time of one single batch truck will cost perhaps $10 a day. There is here, then, what may reasonably be termed a choice of evils for expense is involved no matter which choice is made. In a general way, the higher the efficiency of operation, the less a contractor will need to provide by way of extra truck supply. But, as an efficiency of operation exceeding 90 per cent is astonishingly difficult of attainment, it is thought that, as a general proposition, a 10 per cent excess truck supply is the lowest that it will ordinarily be wise to use.

Another aspect of this case is that inadequate transportation facilities are the rule rather than the exception on paving jobs. Indeed, much of the poor progress made on this work is directly traceable to this cause. Contractors should give more thought to this matter. In the very nature of the case, the force employed on other than hauling work must be maintained at practically constant size without regard to the yardage laid per day. The amount of equipment on the job is, of course, constant. Daily payroll and depreciation charges, therefore, remain about the same, whether production is high or low. On the other hand, varying the number of trucks does not vary the cost per batch hauled. It merely affects the rate of delivery. It, therefore, naturally results that any under supply of transportation sharply increases the cost. On the other hand it is rather a common practice to use all of the transportation all of the time. This

is wholly indefensible. To keep the cost of hauling at a proper level the number of trucks worked should be in close harmony with the number required. A reasonable excess in the truck supply as insurance against the loss of time at the mixer is proper but to keep more than this merely serves to increase the cost of hauling. A brief study of the formula for the time required per trip will show that, except for the influence of the constant, which is of minor importance except on short hauls, the time per trip and therefore the cost of hauling varies directly with the distance hauled. In other words, hauling materials to the third mile will cost about three times as much as hauling them to the first mile if the transportation used is kept in harmony with the work to be done. On the other hand, to work all the transportation all of the time results in making each mile equally expensive until the full distance at which the trucks can supply the mixer is reached! Operation of this sort ruins profits.

The water supply is not commonly so considered but actually is part of the material supply. It is, of course, apparent that anything that interferes with the water supply stops production. The water supply is, therefore, an important consideration. Without going into the matter at length it may be observed that time losses in this field arise from a number of causes. Generally speaking, the most conspicuous of these is moving the hose to another pipe line outlet. As commonly conducted, this operation stops production for from 5 minutes to as much as 10 or 15 minutes. This loss takes place two or three times a day. It can be wholly eliminated by using a double hose connection on the mixer and having two lengths of hose on the job. It can be largely eliminated by careful and systematic work in moving the hose. The latter method was used on this job.

Inadequate water supply often is a cause of trouble. The common practice today on paving jobs is to use 2-in. pipe for carrying the water delivery. This is a leave-over from the days of the 3-bag paver, but for serving a modern 5-bag paver pipe of this size is totally inadequate. A 3-in. pipe is the smallest that can appropriately be used for while the mixer can generally be served through a 2-in. pipe, to do so generates pressures at the pump and in the line which are ruinous. Moreover, one of the outstanding features of most concrete paving work today is the utter inadequacy of the supply of water available for curing. One seldom sees a pavement which, in dry weather, is properly wet down. The cause is apparent enough. The contractor, with his existing 2-in. pipe, simply cannot run his mixer and meet the specifications governing curing.

On this job, preparing subgrade caused a loss of 1.6 per cent of the working day and finishing the concrete caused a loss of 4 per cent of the working day. Other miscellaneous items caused a loss of 0.6 per cent of the day. These last commonly are small difficulties that cannot be foreseen.

Losses of time due to subgrade as well as losses of time due to finishing should never occur. The mechanical means available for handling the subgrade and for handling the finish are so simple and so effective that those operations ought never to affect production. On this particular job,

the underlying difficulty really was erratic inspection, which suggests that there are a few matters in regard to inspection that ought to be more fully understood both by contractors and engineers. One of these is that the specifications should be viewed as a whole. Years ago an outstandingly brilliant professor at the University remarked to the writer:

"It is impossible to learn architecture by taking the bricks out of a wall and studying them with a microscope! Rather, one sits on a hill and meditatively considers the beauty of the structure as a whole."

This is equally true of specifications. Much of the trouble which contractors have in dealing with inspectors arises from the fact that the latter, or the engineer directing him, in looking at the specifications with a microscopic range of vision has brought into this moment's field of vision some one item which he then proceeds to enforce without the slightest apparent conception of its real relation to the general structure which any complete specification actually is! As a result the enforcement of specifications becomes unbalanced. On a job recently studied the inspector told one of the writer's assistants, that as long as the center point was straight none of his bosses paid any attention to the rest of the work! This appeared to be literally true for the finish was rough, many of the batches were wet, no particular attention was paid to how the reinforcing was placed, and the accuracy of subgrade finish left much to be desired, to mention only a few of the conditions prevailing. But let the center joint get a little out of line and there was trouble enough!

Another matter that gives contractors a great deal of trouble is the clause "or to the satisfaction of the engineer" and its many equivalents. Any discussion of efficiency, to be at all complete, must make reference to the fact that the tendency of engineers and inspectors to view such clauses as "carte blanche" to make any requirements they please costs contractors a great deal of money. The modern tendency in writing specifications is to define methods, materials, practices and tolerances with considerable clarity. This practice is open to the criticism that it deprives contractors of the incentive to use their ingenuity in improving these and thereby favorably affecting cost. But while it is not unlikely that the time may come when more latitude will be given contractors and with it more opportunity for constructive thinking in the field of methods and practices, the fact remains that a correct interpretation of existing specifications would not only relieve contractors of much worry and expense but would help toward this end for the clause, "or to the satisfaction of the engineer," both legally and historically, refers only to the engineer's authority in regard to other specified methods, materials, practices, usages, tolerances, etc. The engineer himself commonly has no authority to initiate changes, but rather is required, both historically and legally, to accept and permit the use of practices, etc., which in his judgment are equal to these prescribed whenever the contractor desires to make such substitution. The common attitude among engineers that a new method, different materials, unusual proportions, etc., though under test offering finished work as good

or better than that specified, should be refused, especially if the contractor's profit may thereby be improved, obstructs constructive thinking, needlessly increases cost to the contractor, hurts the engineering profession, and, in all of these ways, acts needlessly to increase the cost paid by the public for a valuable paving material.

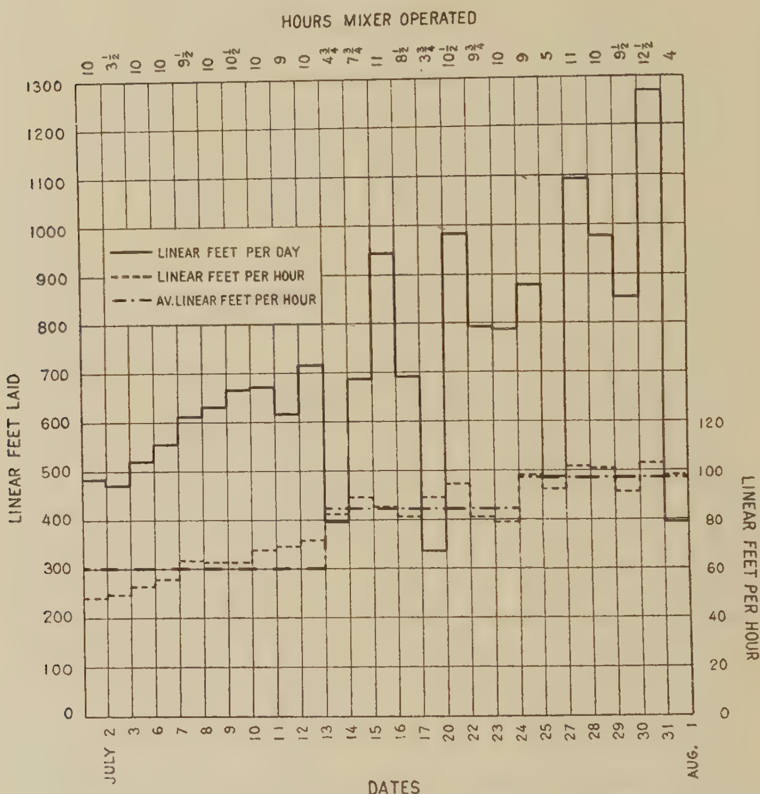

FIG. 3.—DAILY AND HOURLY PRODUCTION ON FEDERAL-AID PROJECT NO. 235, HOWARD CO., MISSOURI; OPERATIONS GUIDED BY STUDIES MADE BY DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS.

The losses in time having been determined and the remedies devised as discussed above, there remains nothing but the actual application of these to the job in question. To change the practices in force on a going job takes time. A month was required on this project to develop a rate of efficiency deemed fairly satisfactory. At first progress was slow but as the men became familiar with the new ideas, they worked diligently toward the desired end with the result that, during the last week of the test period,

production was consistently maintained at a reasonably satisfactory level. The results are shown in Fig. 3.

In closing it may be pertinent to remark that there was nothing unusual about this project. These and other studies indicate that efficiency in production is not a matter governed by local or regional conditions. Given good supervision, a trained appreciation of what the elements of supervision are, and proper equipment with which to work, production equal to or better than that secured on this job during the last week of the Bureau's direct contact with it, should be secured on any other job having a good 5-bag mixer, with the necessary correlated equipment, and this without regard to where it is working.

Transverse Testing of Concrete.

H. F. Clemmer* and Fred Burggraf.†

In highway research of the last few years nothing has been more clearly demonstrated than the instability and non-uniformity of the pavement subgrade. For this reason the pavement must be designed as a floor slab rather than a wearing surface. Therefore, practically all of the formulas developed for highway design in the last few years use the transverse strength as a factor rather than the compressive strength. However, as this change in design has been comparatively recent, tests of materials used in pavements, as well as the data compiled concerning these materials, and the finished concrete, have dealt with the compression test. There has been considerable discussion, with little knowledge gained, as to the measure in which the ultimate compressive stress indicates the transverse strength of the concrete.

The compressive strengths of cores drilled from the same sections of concrete pavements have varied as much as 150 per cent of the minimum. This non-uniformity of the strength of cores drilled from constructed pavements caused the U. S. Bureau of Public Roads to question as to whether or not the strengths of the pavements from which the cores were taken were actually as non-uniform as was indicated by the results of the compression tests. As this non-uniformity was thought to result from variations in the method of compression testing an attempt was made to divide these variations into classes and to correct them as far as possible. However, later in the investigation it became apparent that conditions causing these differences were such an integral part of the test that their correction was highly improbable.

The first class of variation resulted from the type of testing machine used in making compression tests which, though the movement of the load was uniform, the application of the load was not uniform and therefore the stress at any one interval of time was in no way related to the stress at another interval of time. This resulted in a non-uniform rate of application of load between various cores due to the difference in their elasticity. The error produced through the use of this type of machine can neither be corrected nor its effect computed.

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†Assistant Engineer of Materials, Illinois Highway Division.

The second class of non-uniformity of compression tests results from variations in the methods of capping of the cores. In an attempt to correct this error several investigators have advanced methods intended to standardize the process of capping. In the Illinois Highway Laboratory it was thought possible to eliminate capping altogether by sawing off the ends of the cores and in this manner obtain smooth, parallel bearing surfaces, which would insure a uniform distribution of load. However, after installing a new saw and using this method on approximately one thousand cores, the results showed that not only did the non-uniformity still exist but the cores tested by this method showed a lower strength than those which were capped. In Table I is given the results of various methods of capping

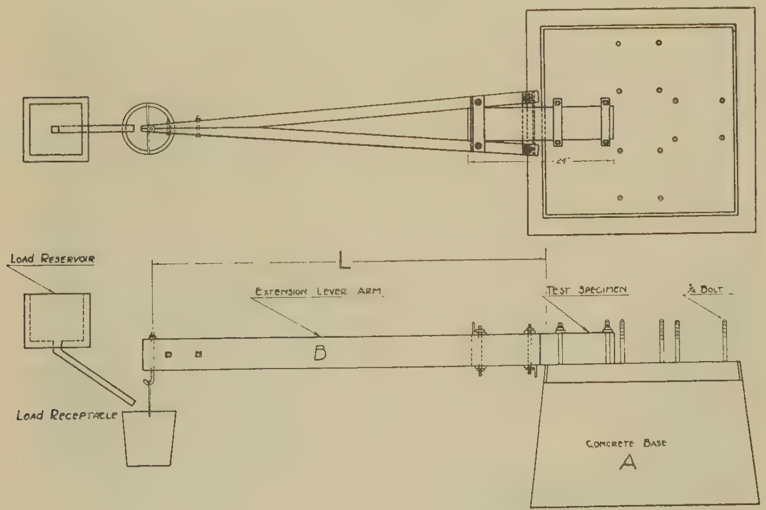

FIG. 1.—CANTILEVER TRANSVERSE TESTING APPARATUS—DETAILS.

on the compressive strengths of cast cylinders. The detrimental effect of sawing off the ends of cylinders to the depth that the cross-sectional area of the coarse aggregate is exposed is shown. Also, the effect of no capping on cylinders, which were only trowelled after casting, is given. In Table II is given the compressive strength of 700 sawed road cores and 700 capped road cores. It will be noted that the average reduction was nearly 900 lb. per sq. in. or over 30 per cent. Further tests, made in an effort to discover a reason for these variations, showed that the moduli of elasticity of the aggregates used in the concrete were in all cases greater than that of the matrix, that the exposed aggregate took the load even though the ends of the core were smooth, and that, therefore, the hard pieces of aggregate were forced axially into the core in such a manner as to cause a wedging or splitting action. For this reason cores, in which the modulus of elasticity

of the aggregate approached that of the matrix, had a much more uniform distribution of the load and showed a greater ultimate strength. Also the variation in elasticity between the aggregates on various sides of the cores resulted in a difference of deformation, for sides of the same core, while under load.

A third cause of variation is the impossibility of eliminating friction as an indeterminate factor in the compression test. The friction between the head of the testing machine and the end of the core is so great that the end of the core is not allowed to expand as the core is compressed and for that reason the core is restrained to some extent from breaking.

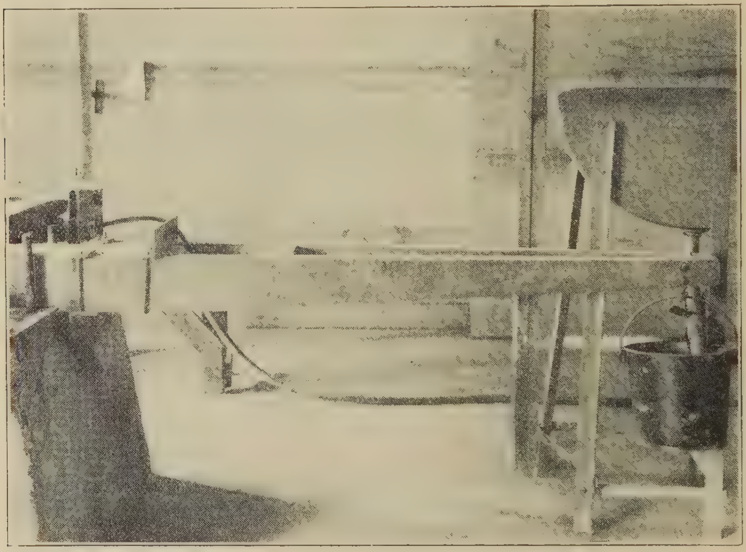

FIG. 2.—APPARATUS SET UP IN LABORATORY.

Attempts to correct this action by placing a layer of expansive material (rubber) between the end of the core and the head of the machine failed because the material expanded under load to such an extent that the end of the core was pulled apart before the ultimate strength of the core was reached.

As a means of comparison between the laboratory specimens used in the study of the compression test, and for field testing for which the compression test is not economical, a transverse testing machine has been developed by the Illinois Highway Testing Laboratory. The beams are supported as cantilevers and an extension arm with a container at the extreme end placed over the free end of the beam. Load is applied by an even flow of shot or water from a separate container equipped with a quick-acting

TABLE I.

End Condition of Cylinder	1 : 2 : 3½ Mix	
	7-day	28-day
Capped Lumnite Cement	1195	2285
Capped Neat Cement	1120	2280
Capped Plaster Paris	1130	2230
Trowelled Mortar Surface	905	1850
One End Sawed Off	890	1780
Both Ends Sawed Off	705	1760

NOTE.—Each result average of 15 cylinders.

TABLE II.

CORES SAWED		CORES CAPPED	
Mark	Ave. Comp. Stress	Mark	Ave. Comp. Stress
B.1 to 44	2768	B.700 to 713	3313
45 to 78	3362	714 to 741	3518
79 to 100	3957	742 to 769	3203
101 to 123	2909	770 to 797	3284
124 to 151	2627	798 to 825	2647
152 to 179	2692	826 to 853	3006
180 to 208	2863	854 to 881	3897
209 to 236	2999	882 to 909	4130
237 to 264	2939	910 to 937	3669
265 to 292	2887	938 to 965	4211
293 to 320	3047	966 to 993	4788
321 to 348	3240	994 to 1021	4610
349 to 376	3165	1022 to 1049	4172
377 to 404	2982	1050 to 1077	4547
405 to 432	3155	1078 to 1105	3765
433 to 460	2535	1106 to 1133	3376
461 to 488	2600	1134 to 1161	3621
489 to 516	2349	1162 to 1189	3003
517 to 544	1663	1190 to 1217	3583
545 to 572	1912	1218 to 1245	2972
573 to 600	2376	1246 to 1273	3217
601 to 628	2468	1274 to 1301	3735
629 to 657	2721	1302 to 1329	3727
658 to 685	3004	1330 to 1357	3851
686 to 699	2847	1358 to 1385	3849
		1386 to 1400	3657
Average	2802		3666

Note.—30.8 per cent increase in strength by capping cores.

TABLE III.—COMPARISON OF TRANSVERSE AND COMPRESSIVE STRENGTHS AS THE REVEALING MEDIUMS OF THE QUALITY OF CONCRETE.

Transverse Strength			Compressive Strength		
Lb. per sq. in.	Diff. bet. Max. & Min. Str.		Lb. per sq. in.	Diff. bet. Max. & Min. Str.	
	Lb. per sq. in.	Per cent		Lb. per sq. in.	Per cent
400			1008		
398	2	0.5	1810	802	79.6
430			2000		
447	17	0.4	1280	720	56.2
387			1352		
376	11	0.3	2165	813	60.1
415			3520		
439	24	5.8	2140	1380	64.5
467			3700		
479	12	2.6	2100	1600	76.2
564			2610		
551	13	2.4	1770	840	47.9
389			2530		
353	36	10.2	1065	1465	138.5
518			2520		
529	11	2	1555	965	62.1
479			2760		
465	14	0.3	1610	950	52.6
441			2900		
456	15	3.4	1400	1410	94.6
438			3040		
416	20	5	1480	1560	105.4
559			2150		
552	7	1.3	3460	1310	60.9
530			2005		
526	4	0.8	2940	935	46.6
518			3400		
455	23	5	1720	1680	97.7
614			4150		
573	41	7.2	3040	1110	36.5

Average Difference—3.7% Average Difference—72.0%

TABLE IV.

Trans. Str. Lb. per sq. in.	Comp. Str. Lb. per sq. in.	Ratio T/C in Per cent	Trans. Str. Lb. per sq. in.	Comp. Str. Lb. per sq. in.	Ratio T/C in Per cent
435	2582	16.8	504	2779	16.9
434	2599	16.7	470	3249	14.4
437	2484	17.6	463	3085	15.0
420	2620	16.0	487	3011	16.1
420	2402	17.4	472	3143	15.0
379	1943	19.5	452	2805	16.1
396	2063	19.1	480	2835	16.9
418	1852	22.5	481	2394	20.1
384	2167	17.7	481	2445	19.6
395	2119	18.6	494	2669	18.5
509	3075	16.5	496	3680	13.5
521	2872	18.5	490	2920	16.7
563	3024	18.6	491	3151	16.1
483	3042	15.8	541	3186	16.9
563	2932	19.2	500	3375	14.8
545	3184	17.1	512	3329	15.3
486	3291	13.4	513	3359	15.2
519	3622	14.3	491	3580	13.7
496	3291	15.3	520	3342	15.5
530	3045	17.4	467	3373	13.8
447	2634	16.9	505	2760	18.3
466	2827	16.4	513	2878	17.8
477	2852	16.7	484	2372	20.4
459	2312	19.8	489	3149	15.5
479	2563	18.6	535	2587	20.6
		Average	490	2859	16.98

Note.—Each value the average of ten tests. (28-day period.)

valve. (See Figs. 1 and 2.) Calculations include overhang of specimen, extension arm and the weight of shot or water required to cause failure. By this method a uniform application of load is made possible and, further, more than one break may be obtained for a check. Exact coincidence of results is not uncommon with the use of this machine while results on the same specimen rarely vary over small percentages. To compare results of this type of testing with those of the compression test, 700 beams, 6 x 8 x 30 in., were broken in two places with the cantilever testing machine and cores drilled from the three remaining pieces tested in compression.

FIG. 3.—BEAM SPECIMEN AFTER BEING TWICE TESTED, AND CORES DRILLED FROM REMAINING SECTIONS.

Thus three tests in compression and two in flexural strengths were obtained for each specimen beam. (See Fig. 3.) In Table III is given a tabulation of results of 15 specimens chosen at random from 200 beams tested in the first series. In the case of both the compressive and transverse strengths the percentage difference is calculated on a basis of the minimum strength observed. In Table IV is given the results of 500 tests of transverse and compressive strengths on 1 : 2 : 3½ mixed concrete with different coarse and fine aggregates and the ratio of transverse to compressive strengths.

In addition to the accuracy of the cantilever testing method another factor of interest is its great economy. Heretofore, testing of concrete has comprised a series of highly technical methods requiring costly machines

and equipment only available in well equipped laboratories. The introduction of the cantilever machine has reduced the cost of equipment and machinery required to a negligible amount, made possible the testing of specimens in the field, and materially increased the accuracy of the test.

The adaptability of this method to field conditions has proven of great value to the Illinois division. Economically, it has become more and more imperative that pavements be put in use as soon after construction as possible and that some means be found of completing pavements during the construction season even though low temperatures exist. The cost of pro-

FIG. 4.—SPECIMEN CLAMPED TO TRUCK AND READY FOR TEST.

viding and maintaining suitable detours for traffic during construction periods is a very considerable item in any highway construction program not taking into account the enormous cost to the public of using these detours rather than the finished slab. Furthermore, many sections of various routes are left unpaved when, due to adverse weather conditions in the fall of the year, it is not possible to complete the work. This situation has been relieved in several instances by the use of an accelerated curing method closely controlled through field strength tests by the new cantilever method.

In Illinois two accelerated methods have been used to cope with the difficulties already mentioned. These are (1) the use of calcium chloride

applied to the surface of the slab as a curing agent or the incorporation of the material in the mix as an admixture, and (2) the use of lumnite cement.

The use of calcium chloride applied on the surface as a curing agent has been found, from field tests made with the cantilever apparatus, to produce proper curing in half the time required by the old methods. However, when it is necessary to open a pavement to traffic as soon as possible or to pour concrete under low temperatures much more satisfactory results may be obtained with the use of calcium chloride as an admixture.

In order to carry on construction with accelerated curing it is necessary to have some method of test which can be easily used in the field to ascertain the strength of the concrete at increasing ages, to determine the

FIG. 5.—SPECIMEN CLAMPED TO BRIDGE AND READY FOR TEST.

intensity of the treatment necessary and finally to indicate when the slab is strong enough to carry traffic. It is particularly important to determine the increase in strength as different brands of cement respond in different degree to calcium chloride as an admixture. Therefore, the success of these methods depends entirely on the correct analysis of the problem in the field and subsequent checking by some accurate field test. For this purpose specimen beams are made in easily constructed forms of the same concrete used in the slab. These specimens are allowed to cure in the same manner as the slab and are broken at regular intervals in a testing machine which for the most part may be constructed on the job. In these field tests the specimens are held in cantilever by being clamped to the rear end of a truck or some nearby structure such as a bridge. (See Figs. 4 and 5.) The cantilever arm with its load container is then attached to the beam and the load applied. The loading medium generally

used is water applied by the mixer hose line, however, dry sand, shot or any free running material may be used. When the results of the tests show the strength of the concrete has increased to that used in the design formula it is definitely known that the pavement may be subjected to traffic without danger of failure. Or, when the concrete is laid during low temperatures it will be definitely ascertained that the proper strength has been gained. Illinois has opened some pavement for traffic in as short a time as 96 hours which would not have been advisable had it not been determined by means of the cantilever testing machine that the concrete had gained sufficient strength to withstand the loads.

It may also be seen that the use of this machine will solve many of the field problems of the materials engineers, for by its use the effect of questionable aggregates, cement, or weather conditions may be easily and accurately determined under the same conditions as the actual construction.

DISCUSSION.

JOHN TUCKER, JR. (*By Letter*).—The problem presented in the present paper is of statistical interest, dealing with the relative strength dispersion of two dissimilar structural elements made of one material. The two forms, however, measure two different strength qualities—pure tension in the beam, and the usual compound shear and compression failure exhibited in the test cylinder. The test beam, measuring the tensile strength of the material, is of value only as indicating the merits of that material to resist tensile stresses. To use the test beam as an indicator or check upon the value of the material in compression would require exhaustive tests to establish the true correlation between the tensile and compressive strengths, and to prove the validity of the relationship independent of all types and selections of sand, gravel, cement, water ratio, etc. Mr. Tucker.

In pavement design tensile strength is of great value, far more so, according to Messrs. Teller and Pauls* than the compressive strength, and in this case the beam is the ideal test specimen. Furthermore, the existence of a definite relationship between the tensile and compressive strength of the material, is in this case of no importance whatsoever.

The writer has made a study of the strength dispersions of compressive cylinders of varied diameters, and has been successful in developing a mathematical relationship between the magnitude of the dispersions and the cylinder diameter. The problem of the present paper, to show the relationship between the strength dispersion of different qualities of the material, as well as for elements of dissimilar form, is far from difficult of solution upon a theoretical mathematical basis.

The basis of computation of the ultimate tensile stresses in the beams has not been given in the paper, the numerical values seeming to indicate the computation upon the assumption of the neutral axis at mid-beam, whereas the computation should be based upon the actual position assumed by the neutral axis, as experimentally determined. The position of the 8-in. dimension of the beam as tested, and the diameter of the compression test cylinders have not been given in the paper.

In the study of a problem such as the present, the measure of the dispersion should be given as the standard deviation, δ, and not by the maximum and minimum values. The standard deviation is given by the equation:

$$\delta = \sqrt{\frac{\Sigma v^2}{n-1}}$$

where

v is the variation of the individual specimen from the mean value; and

n is the number of specimens.

*Paper, "Concrete Pavement Design," J-314, this volume.

Concrete Pavement Design.

By L. W. Teller* and J. T. Pauls.**

Introduction.

One of the most outstanding developments in this country during the past few years has been the phenomenal growth of motor transportation. The production of low cost motor equipment and cheap motor fuel have created a demand for improved highways. The highway engineer has been faced with the urgent necessity for the design, construction and maintenance of a sufficient mileage of highway, of a type best suited to the conditions of location, traffic and funds available.

To meet this insistent demand many types of pavement have been built, among the most important of which is that of portland cement concrete. Unfortunately, research has not kept ahead of construction, and many pavements have been laid down without a proper appreciation of the many factors affecting the design, and often the result has not been satisfactory. The intensive highway research of the past few years has supplied much information hitherto lacking, so that we are today much nearer to rational concrete pavement design than we have ever been. It will be evident that our methods are not entirely rational and that there is a real need for continued highway research.

The problem of pavement design at once resolves itself into two parts; the first, that of forming a correct estimate of the forces which will act upon it; the second, that of so designing the structure of the pavement that it will resist these forces with maximum economy. In other words, the same two fundamentals that are met in every problem of structural design are involved in the design of a concrete pavement, and in the study of pavement design we will find that the difficulty is largely with the first of these problems due, in a great measure, to the influence of the subgrade.

The Subgrade.

The pavement, as a structure, must depend upon the underlying subgrade for support. The character and condition of this subgrade have been shown, both by experience and by research, to greatly affect the behavior of the pavement throughout its life. Therefore, it becomes most important to

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acquire some knowledge of the various properties of the materials which comprise the foundation upon which the concrete road and its burdens must be carried.

Consider for a moment the source of these materials, and it becomes apparent why they vary so widely in their characteristics. Originating from parent rocks differing greatly in composition, broken down by the destructive forces of nature acting over long periods of time, carried and deposited by wind, by water, by ice, subjected to physical and chemical agents of every sort, the final product may be anything from solid rock to a finely divided colloidal material. The physical properties of these materials vary at least as widely as their structure, yet it is over these soils that the pavements must be laid, so that their study, however complex, is imperative.

The subject of subgrade soils has received a great deal of attention in the past few years, and this research has yielded certain laboratory and field tests and methods of study which enable the highway engineer to make a more intelligent estimate of the foundation conditions for the pavement he is to build.

For the purposes of soil identification, it is now generally conceded that the methods and nomenclature developed through years of experience by the Bureau of Soils of the Department of Agriculture are the best. A very considerable part of the United States has been accurately mapped and these soil maps are already available. Soil experts can furnish not only exact identification, but much valuable information as to the drainage characteristics of the various soil types as well.

During the past five years the U. S. Bureau of Public Roads has been developing standard laboratory tests for subgrade materials with the view toward determining their field behavior. These tests were intended to indicate certain characteristics of the various materials which might influence their behavior under a pavement. Any detailed description of these tests would lengthen this discussion unduly, but a brief description will serve to emphasize those properties which at present seem most important from a design standpoint.

The dominant characteristics of a good subgrade appear to be:

1. Uniform structure.
2. High percentage of crystalline material.
3. Good internal drainage.
4. Low volume change.
5. Low moisture retaining capacity.
6. High bearing value.

To study these and other possible properties, the following tests have been developed:

1. Mechanical analysis.
2. Dye absorption.

3. Moisture equivalent.
4. Percentage of capillary moisture.
5. Shrinkage value.
6. Moisture retaining capacity.
7. Comparative bearing values.

Mechanical Analysis.—The method employed is a modification of that used by the Bureau of Soils. The classification depends on the size of the particles, as follows:

Classification	Size Limits of Particle Millimeters	
1. Coarse material	Above	2.000
2. Fine gravel	2.000	1.000
3. Coarse sand	1.000	0.500
4. Medium sand	0.500	0.250
5. Fine sand	0.250	0.100
6. Very fine sand	0.100	0.050
7. Silt	0.050	0.005
8. Clay	0.005	0.000

The silt and clay are removed by washing, regulated by microscopic examination. The percentages of material of each size larger than silt are then determined by sieve analysis. The silt and clay are separated by centrifuging, control of this process being through frequent microscopic measurement of the suspended particles.

One difficulty which has always attended the interpretation of laboratory tests of subgrade soils, has been the apparent lack of close inter-relation of the various properties. It has seemed that many of the characteristics of the materials should be a direct result of the grading of the soil. It has been suggested that, as it appears that the finely divided particles are the cause of much of the so-called "bad" subgrade materials, we must determine more closely the grading of these very fine particles. Soil physicists have attempted this, but so far the methods for making this determination are in more or less of an experimental stage and are merely mentioned here as a development which may yield most valuable results.

Dye Adsorption.—Soils which contain very finely divided material have the power to de-colorize aniline dye solutions which pass through them. This property is, then, an indication of the amount of this finely divided material which is present in the soil. It also appears to be true that these same fine materials make for plasticity, an extremely undesirable characteristic in a subgrade material. There are at present difficulties in the interpretation of these dye adsorption values, but it may be stated that, in general, a high dye adsorption number is indicative of a poor subgrade material.

Moisture Equivalent.—This test was developed some time ago by soil physicists and consists of a determination of the moisture retained by a saturated sample after it has been subjected to a centrifugal force of 1,000 times that of gravity for a definite period of time. Various soils differ greatly in this respect, and apparently this test is most significant as a measure of the ease with which the soil under consideration may be drained. Sandy soils retain little or no moisture; very plastic clays may retain over 50 per cent of their moisture under this test. A very simple "field" moisture equivalent test has been developed which appears to check the laboratory method quite closely over the central part of the range of moisture equivalent values. This test is described elsewhere.[1]

Capillary Moisture.—A laboratory determination of the amount of moisture which a soil may absorb through capillary action gives important indications as to the field moisture to be expected under a pavement, due to the capillary lift from the water table below. Soils having low percentages of capillary moisture are desirable for subgrades.

Shrinkage Values.—One of the most important characteristics of all subgrade materials is the tendency to change in volume under a change in moisture condition. Sandy soils change very little, but the volume of certain clays may change as much as 50 per cent or more. Under a pavement the moisture conditions are not uniform. Test borings have shown the moisture content near the edge to be quite different from that in the center of the pavement. In a soil whose volume change (or shrinkage value) is large, this will obviously lead to uneven support or very probably to no support at all under the edges of the pavement. Actual, measurable separation between the slab and the subgrade occurs and it is not uncommon to find many longitudinal cracks in pavement laid on this type of subgrade. It is most desirable, therefore, to have as a subgrade a material whose volume change is small, and this laboratory determination of the shrinkage value serves to measure this important property.

Moisture Retaining Capacity.—This is a laboratory determination of the maximum amount of moisture which the soil is capable of retaining. It is another measure of the drainage tendencies of the material, being closely allied to the moisture equivalent test. For this reason it has been discontinued as a routine test. As a general rule, the higher the moisture capacity of the soil, the less desirable the material is for a subgrade.

Comparative Bearing Value.—Many attempts have been made to devise some field or laboratory test which would indicate the relative bearing value of various soils. The problem is extremely complex and the results of such research as has been done have not been entirely satisfactory. These are indications that soils do have a very definite bearing value, but further research is needed to properly interpret this test.

[1] Practical Field Tests of Subgrade Soils, by A. C. Rose. *Public Roads*, Vol. 5, No. 6, August, 1924.

All of these tests have been described in detail elsewhere.[2]

Turning now from the testing laboratory to the field, let us estimate the situation. We know that almost any material, if confined against displacement and kept dry, will make a good subgrade. An example of this is found in the brick roads of Florida. Here a loose, dry sand is used to carry a brick pavement simply by confining it at the edges. In this case the very nature of the material supplies the drainage. But this is an exceptional condition. Usually it is not necessary to supply curbs to confine the subgrade, but the problem of drainage becomes of increasing importance as the type of subgrade material changes from sand to clay.

All of the data so far submitted indicate that so far as this country is concerned, the character of the clay remaining constant, good subgrade soils have a low clay content by mechanical analysis and bad subgrade soils are characterized by a high percentage of clay.[3]

Certain exceptions to this have been pointed out by H. H. Bennett,[4] who says further that

"Clays of uniform red, brown and deep yellow colors are better oxidized than those of whitish, grayish, bluish and pale yellow colors, or mottlings of these colors, and they almost invariably are better drained, shrink less on drying, swell less on wetting and are much firmer (less miry) in the presence of excessive moisture.

"Water passes slowly through the light-colored and mottled plastic clays; they become saturated with penetrating moisture and eventually soft and unstable."

These soils which are so difficult to drain present the chief subgrade problem which the highway engineer is called upon to solve, for, as has been said before, if subgrade materials can be kept relatively dry, little trouble will be experienced with them.

Water enters the subgrade in a variety of ways. It should be emphasized that it is the moisture in the soil immediately under the pavement with which we should be most concerned. Often it is largely surface water which is not properly cared for and seeps into the subgrade around the edges and through the cracks and joints of the pavement slab. Another source is that of water vapor which, rising through the soil pores, becomes condensed against the lower surface of the pavement and drops back as free water to saturate the upper portion of the subgrade. Other moisture is being continually drawn into the subgrade from the surrounding soil by the capillary action of the material itself. Then, again, there are often porous strata which at certain seasons of the year become water-bearing and carry water into the subgrade where, no exit being provided, it collects and a bad subgrade condition results.

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[2] Physical Properties of Subgrade Materials, by J. R. Boyd. *Proceedings*, American Society for Testing Materials, Vol. 22, 1922.

[3] Researches on the Structural Design of Highways by the U. S. Bureau of Public Roads, by A. T. Goldbeck. *Transactions*, Am. Soc. C. E., Vol. 88, p. 264, 1925.

[4] Clay Soils in Relation to Road Subgrades, by H. H. Bennett, soil scientist. *Public Roads*, Vol. 6, No. 8, October, 1925.

In building a pavement over any subgrade material whose characteristics are such that an increase in moisture results in a marked decrease in stability, exceptional precautions must be taken, either to keep the moisture content low or to change the character of the material.

As a general rule, any treatment of the subgrade which would increase the stability of an earth road, over the same location, will improve its stability as a subgrade. This is the thought behind the so-called "progressive method" of road building, where by gradually improving and stabilizing the earth roads, an ideal subgrade is provided against the time when a high type pavement will be needed to carry the traffic on the same location.

Some of the treatments which have been used in the past for the improvement of bad subgrades are:

1. The use of granular materials as a layer immediately under the pavement.
2. The admixture of various materials with the subgrade soil.
3. Increasing the depth of the side ditches.
4. The use of drains parallel to and along the edges of the pavement.
5. The provision of drains to intercept all porous strata which may become water-bearing.

Granular material added as a blanket or layer immediately under the pavement is being tried in a number of states, both as regular practice and as an experiment. There seems to be considerable virtue in the treatment, which is usually not unduly expensive and is not difficult to apply. This method needs careful design and supervision, however, as it is most important that thorough drainage be provided at all times for this porous layer. Otherwise, it will simply collect excessive moisture and the bad condition will be further aggravated.

A number of attempts have been made to improve unstable subgrades through the admixture of various materials such as sand, portland cement and lime. These experiments have usually resulted beneficially, although somewhat costly and presenting construction difficulties. This treatment aims to so alter the material in the subgrade that it will have low volume change and good supporting power under existing moisture conditions. Fig. 1 shows some interesting experimental data which indicate the effect of admixtures in various amounts on the volume change of a bad clay. This work is further described elsewhere.[5]

Increasing the depth of the side ditches will insure the carrying away of surface runoff, especially during spring thaws, and will tend to reduce the normal moisture content.

Drains along the edge of the pavement will, if kept free, effectively prevent surface water from entering the subgrade. It is obvious that the

[5] The Present Status of Subgrade Studies, by A. C. Rose. *Public Roads*, Vol. 6, No. 7, September, 1925.

difficulty will lie in keeping these drains open, particularly in very cold climates.

When cuts are made across the natural stratification, it often occurs that a porous stratum overlying an impervious layer will be uncovered. Such a condition, if not properly cared for, is a likely source of future trouble because, during the spring thaws or following a period of heavy precipitation, this porous layer may become water-bearing. Hence it is important that it be intercepted with a drain before it comes under the pavement.

Attempts have been made to meet bad subgrade conditions by building a stronger pavement, but this practice does not get at the source of the

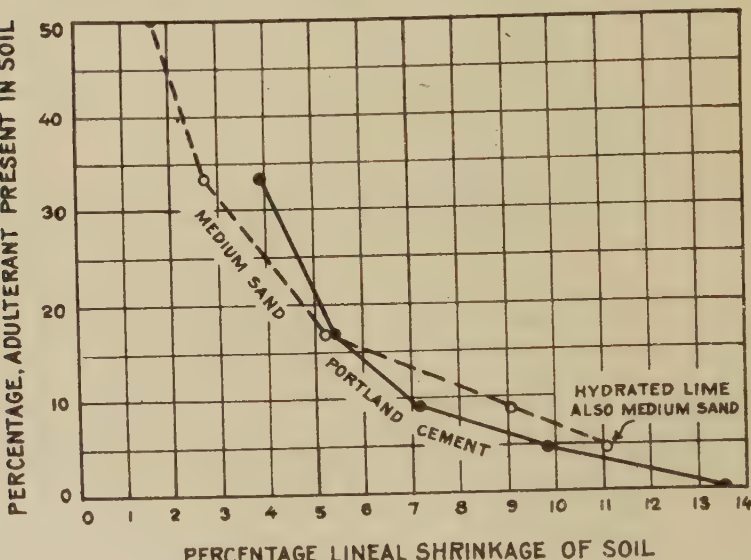

FIG. 1.—EFFECT OF VARIOUS ADMIXTURES ON COVE CLAY.

trouble and it seems probable that as more is learned of subgrade materials and subgrade treatments a better solution will always be possible at less cost.

This brief discussion of the situation regarding subgrades as they affect pavement design will simply emphasize the complexity of the problem and the need for further research. Progress has been made and some remedies have been found which may be applied in practice, but the average subgrade as we know it today must be considered as a foundation which cannot be depended upon to furnish good uniform support to the pavement at all times, because its physical state is constantly changing under variations of moisture and temperature.

CHARACTERISTICS OF THE CONCRETE IN THE PAVEMENT.

If an engineer is to successfully design a structure he must first thoroughly acquaint himself with the properties and characteristics of the materials which he expects to employ in his design. This is just as true for a concrete pavement as for a steel bridge, yet it is a fact which too often has been overlooked.

Portland cement concrete has certain definite characteristics which govern its behavior in a pavement when it is subjected to the various natural forces and to those of traffic. These characteristics can best be illustrated by the study of a concrete pavement from the moment the concrete is placed until after the road has been opened and traffic comes upon it.

Practically as soon as the concrete is in place it begins to harden or "set up." Although it has little strength, it is called upon to resist certain forces. The surface of the concrete begins to dry out and this evaporation is aggravated by wind or hot, dry weather. This loss of moisture produces shrinkage in material which has as yet developed practically no strength. The result is surface checks or hair cracks whose significance should not be under-estimated. It is extremely important, therefore, that the surface of the pavement be kept moist from the earliest possible moment.

Moisture may also be lost from the bottom of the slab through the "blotting" action of a dry subgrade. This moisture passing into the subgrade, in addition to causing shrinkage in the concrete, may result in considerable volume change in the soil itself with consequent strain to the pavement. An example of the effect of this type of moisture loss to the subgrade in the loess soil of Iowa is described by R. W. Crum,[6] who states that the use of tar paper over the subgrade was used to combat this tendency with considerable success in the case described.

In any event, it is important to have the subgrade thoroughly moist at the time the concrete is laid; a superficial sprinkling does little if any good.

If shrinkage of the concrete occurs, the slab as a whole tends to move on the subgrade. This movement is resisted by the forces of friction which, if allowed to develop, will soon exceed the small tensile strength of the green concrete. For this reason the pavement must be kept wet during its curing period. This should be for a period of several weeks, although usually two to three weeks is allowed. The pavement is then uncovered and allowed to dry out, the material contracts and movement over the subgrade begins. As previously remarked, this movement is resisted by the friction developed between the pavement and the subgrade. If the pavement has been divided by expansion joints at frequent intervals, this contraction is taken care of; if not, the tensile strength of the concrete will be exceeded and transverse cracks will be formed to relieve this tension.

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[6] Tar Paper on Loess Subgrade Lessens Hair Cracks in Concrete Pavement, by R. W. Crum. *Public Roads*, Vol. 6, No. 6, August, 1925.

The higher the tensile strength which has been developed up to this time, the less frequent will be these cracks.

Another phenomenon which takes place more or less from the time the pavement is laid is that of warping. This is due to a temperature differential between the upper and lower surfaces of the pavement. The upper surface responding to variations in temperature of the air expands and contracts. The change in temperature under the slab is far less and the result is a flexing of the slab. This causes bending stresses due to the weight of the slab itself. As the warping is caused by a difference in temperature between the top and bottom of the pavement slab, such curing methods as will reduce this difference will help to overcome the possible weakening of the slab due to warping.

The same agencies which cause contraction of the concrete also cause expansion and, while ordinarily the early compressive strength is high, it is possible that excessive compression between two slabs might cause sufficient transverse tension to crack the pavement longitudinally.

Traffic coming upon the road introduces heavy wheel-loads at all points of the pavement slab. These may move slowly and smoothly along the pavement or they may be accompanied by severe impact. Such loads cause heavy bending stresses to be set up and often the time elapsing between these load applications is small.

Finally, as time passes the action of traffic and the natural agencies of temperature and moisture tend to break down the surface of the pavement, so that the material must be dense and resistant to these attacks.

We have, in this brief consideration, indicated certain properties of concrete which it is necessary to study somewhat in order to proceed with the design of a pavement. These characteristics might be summarized as follows:

1. Expansion and contraction due to changes in moisture.
2. Expansion and contraction due to changes in temperature.
3. Tensile strength.
4. Resistance to bending.
5. Compressive strength.
6. Modulus of elasticity.
7. Resistance to repeated stress.
8. Resistance to surface wear or disintegration.

It will be well to review some of the important indications of our present knowledge concerning these various properties of concrete for pavement purposes.

Expansion and Contraction of Concrete due to Changes in Moisture.— It is a rather well-known fact that concrete possesses the property of expanding and contracting under changes of moisture conditions. This has been established by various experiments, among which are those conducted by the Bureau of Public Roads some fifteen years ago. Fig. 2 shows some of the data obtained in this investigation. Other experiments have, in

general, substantiated these results and also emphasized the effect of various cements, aggregates and other factors upon the amount of this expansion. Apparently the concrete remains expanded so long as it is kept wet and begins to shrink as soon as the moisture content is decreased. In the tests mentioned the maximum expansion was 0.01 per cent and the shrinkage was about 0.05 per cent upon complete drying out of the specimen. Another important fact brought out was that the presence of steel rein-

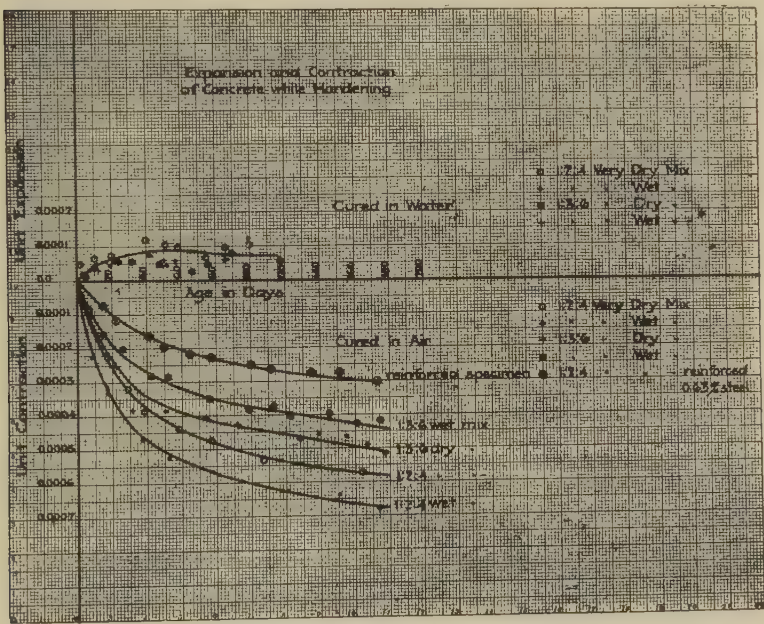

FIG. 2.—EXPANSION AND CONTRACTION OF CONCRETE.

forcement reduces this expansion and contraction. This will be again referred to later.

The effect of this shrinkage on a pavement is to cause tensile stresses to be set up which may exceed the strength of the concrete and cause cracks. It is most important, therefore, to reduce the contraction to a minimum and to delay its occurrence as long as possible in order that the concrete may develop its strength.

Expansion and Contraction of Concrete due to Temperature.—It is also well-known that concrete, like most other materials, expands when its temperature is raised and contracts when its temperature is lowered. The exact amount of this change apparently depends on a number of factors, such as mix, age, temperature and the characteristics of constituent mate-

rials. There is evidence that the value of the co-efficient increases somewhat with the richness of the mix and Dr. W. K. Hatt's experiments at Purdue would seem to indicate considerable variation in its value over a moderate temperature range. He found a value of 0.0000040 at 60 deg. F. and a value of 0.0000065 at 150 deg. F. The generally accepted value for design purposes has been 0.0000055. To visualize what this amounts to, let

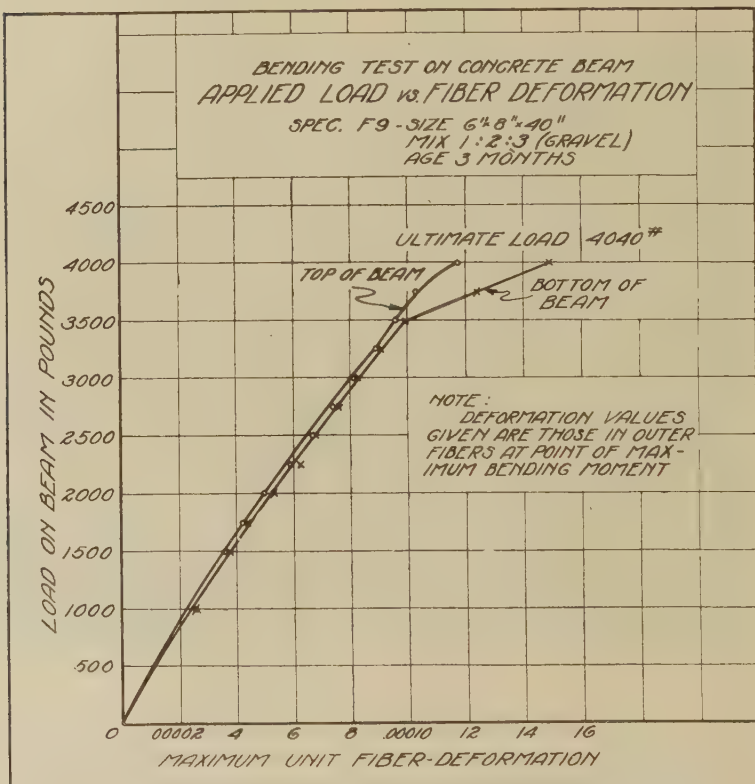

FIG. 3.—MODULUS OF RUPTURE TEST CURVE FOR A CONCRETE BEAM.

us express it in terms of a concrete pavement one mile long, and note that the difference in length of this mile of pavement may be something like two feet from summer to winter. Just as in the case of moisture changes, the result of this will be tensile stresses set up in the concrete.

Tensile Strength.—We have seen how contraction of a pavement due to moisture change or temperature change produces direct tension in the concrete which must be resisted by its tensile strength. If two long slabs

expand toward a common plane such as a construction joint and high compressive stress results in the ends of the two slabs, it is possible for transverse tensile stresses to be set up sufficient to cause short longitudinal cracks to form before any failure by compression occurs. Short longitudinal cracks starting from a transverse crack or joint are very probably caused in this way.

There is need for more experiment on the resistance of concrete to direct tension. What meager data there are seem to indicate that this tensile strength is very roughly 10 per cent of the compressive strength, but this is very unsatisfactory as the effect of the many variables is not known.

Modulus of Rupture.—Heavy wheel-loads passing over the pavement produce bending stresses, as does the weight of the slab when warping occurs. These two conditions often occur simultaneously and to resist them, without cracking, concrete for pavement purposes should possess high resistance to bending, especially at the time the curing is completed. This is a property of foremost importance. Tests for the purpose of measuring this quality have taken a variety of forms and have not been wholly satisfactory. The difficulty seems to have been to obtain check tests on the same concrete. Recently a new method for making this determination has been developed experimentally and seems to give promise of closer check tests. Fig. 3 shows a typical modulus of rupture test curve in which the fiber deformation was measured in both the top and bottom.

Compressive Resistance of Concrete.—Concrete in a pavement will never be stressed to the limit of its compressive strength. Failure in other forms will occur before this can happen. Buckling or "blow-ups" are the form in which a pavement under high compression usually finds relief. Consequently, from a design standpoint compressive strength, as such, is not of great importance. It is, however, a measure of the quality of the concrete and thus serves a most important purpose. It has been the yardstick by which the effect of the many factors which affect the strength of the concrete has been measured. If a concrete for a pavement develops a compressive strength of 3,000 lb. at 28 days, we know that it is of excellent quality and we may reasonably expect that its strength, in general, is of similar quality.

Modulus of Elasticity.—This is a measure of the stiffness of concrete or the ratio of load to resulting deformation. It is commonly determined in compression because of the ease with which it may be done. Such data as are available indicate that for practical purposes its value in the case of tensile stresses may be assumed to be the same as in compression. As a general rule, the better the concrete the higher the modulus value to be expected and the nearer the stress-strain curve will approach a straight line. Pavement concrete may have a modulus of elasticity of over 5,000,000, but a safer assumption for design purposes would be 3,000,000. Con-

crete is not perfectly elastic, but remains in a somewhat deformed condition after the load is released and this deformation gradually disappears with the passage of time.

Resistance to Repeated Stress.—As a wheel-load passes over a slab both tensile and compressive stresses are set up in the pavement, and as the wheel progresses across the pavement these stresses are reversed and finally released. Highways on which concrete pavements are laid are usually heavily travelled, which means that this cycle of stresses may occur several thousand times during twenty-four hours. Research has definitely

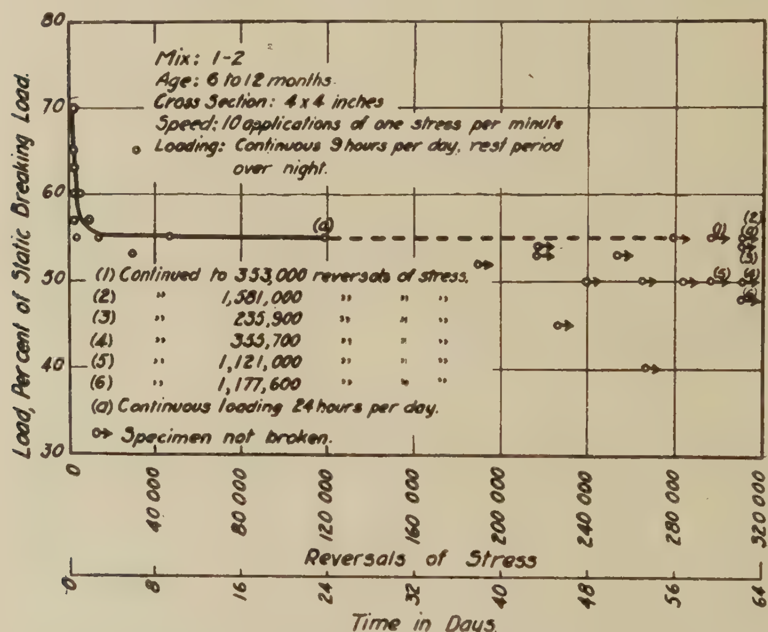

FIG. 4.—FATIGUE ENDURANCE LIMIT FOR CEMENT MORTAR BEAMS.

shown the marked reduction in resistance which concrete has to stress when that stress is repeated frequently. Fatigue in concrete has been the subject of several series of experiments and the results of the several investigations are, in general, concordant. Fig. 4 shows some of the results of tests on cement mortar beams at Purdue University, under the direction of Dr. Hatt, which indicated that, in dry concrete, the effect of fatigue becomes apparent when the stress exceeds 54 per cent of the modulus of rupture. Below this value, apparently, the stress can be repeated indefinitely; above, comparatively few applications may cause failure. If the concrete is in a saturated condition, apparently a somewhat lower value is critical. Concrete pavements are rarely saturated but, conversely, they are

seldom in the dry condition of a laboratory specimen. To the best of our knowledge, then, because of fatigue it would not be safe to design for stresses in excess of 50 per cent of the modulus of rupture where there is likelihood of frequent heavy loads.

Resistance to Wear.—The ability of a good concrete pavement to resist the abrasive action of traffic and the weathering action of moisture and temperature is one of its important properties. To have this resistance, it must be dense and impermeable. Such concrete is obtained only by careful design and construction. Some of the important factors which affect the resistance to wear of concrete pavements have been definitely indicated by the field tests at Arlington, Va., and the conclusions reached are of such importance that they will be extracted from the published report,[7] as follows:

1. That the rate of wear of stone concrete is, in general, not affected by the coarse aggregate provided the coarse aggregate is equal or superior to the mortar matrix in resistance to wear.
2. That excessive wear will result from the use of very soft stone as coarse aggregate, even though used in conjunction with a mortar of satisfactory quality. From the results of these comparative tests, it would appear that stone with a percentage of wear over 7 should not be used in the wearing course of concrete roads.
3. That gravel concrete, in general, is at least as satisfactory from the standpoint of wear as stone concrete.
4. That gravels consisting essentially of siliceous materials are superior as regards both the amount and uniformity of wear to those containing a preponderance of calcareous fragments.
5. That gravels consisting of rounded particles are as satisfactory from the standpoint of wear as those consisting either wholly or in part of angular or crushed fragments.
6. That small amounts of shale occurring in the coarse aggregate will cause both excessive and uneven wear.
7. That the modified abrasion test for gravel in its present form is not an indication of the wear-resisting properties of coarse aggregates. It is suggested that if the severe impact action of the steel balls were decreased much more indicative results would be secured.
8. That blast-furnace slags should prove satisfactory for use in concrete pavements provided the proportion of light, porous slag is so controlled that the weight per cubic foot will be at least 70 lb.
9. That the presence of large amounts of light, porous fragments in blast-furnace slag will cause excessive wear.
10. That somewhat better results are secured by the use of the smaller sizes of slag.

[7] Wear of Concrete Pavements, by F. H. Jackson and J. T. Pauls. *Proceedings*, A. S. T. M., Vol. 24, 1924, p. 895.

11. That slag or stone screenings are, in general, unsatisfactory as substitutes for natural sand as fine aggregates in concrete road construction.
12. That the copper and lead-smelter slags used in these tests would make satisfactory aggregates for concrete road construction from the standpoint of wear.
13. That coarse sands, other things being equal, show greater resistance to wear than fine sands.

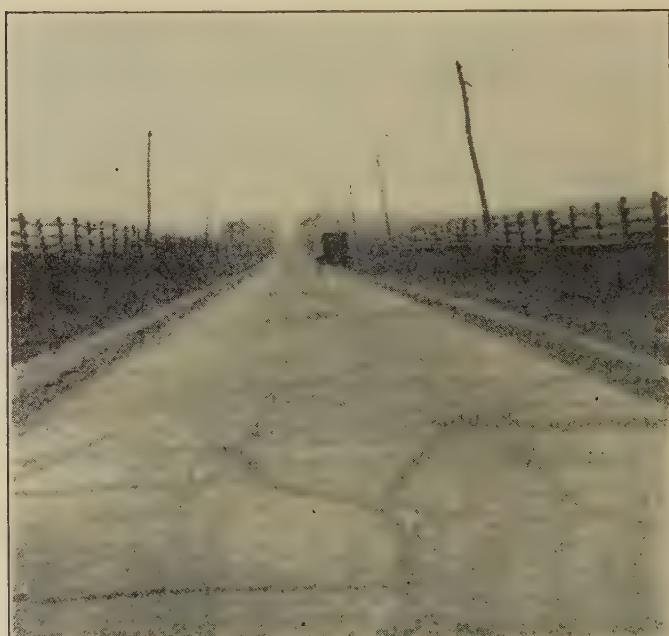

FIG. 5.—TYPICAL CRACKING OF CONCRETE PAVEMENT FROM POOR CURING.

14. That the so-called "tensile-strength-ratio" test is no indication of the wear-resisting properties of concrete made with these sands.
15. That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic.
16. That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties.
17. That the addition of hydrated lime in the proportion used in these tests does not affect the wear-resisting properties of concrete.
18. That so far as resistance to wear alone is concerned, increasing the cement content beyond a cement-sand ratio of 1: 2 does not mate-

rially affect the concrete. Leaner mixes, on the other hand, show marked increases in wear.

19. That unusual precautions should be taken in using mine chats or other similar harsh-working materials, so as to increase workability to a maximum and thus make possible a smoother surface finish.
20. That, other things being equal, either an excessively dry or an excessively wet mix will show less resistance to wear than concrete of medium consistency.

CURING.

In considering the various properties of concrete for pavement purposes, curing has been touched upon. Curing affects all of these properties

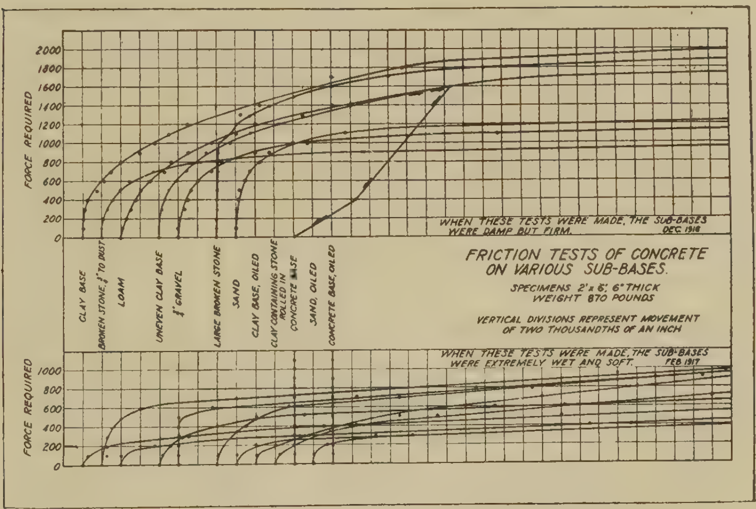

FIG. 6.—FRICTION TESTS ON VARIOUS SUB-BASES.

and through them the life of the pavement. It is of vital concern to the designing engineer, because it determines what strength values he may safely assign to the material he employs.

It can be shown by a simple calculation that the cracking which occurs in the pavement at the time when contraction is allowed to proceed is in direct proportion to the tensile strength which has developed at that time. If proper curing has been neglected, a pavement may fail before it ever carries traffic. Instances of this are not rare. Fig. 5 is typical of this effect and is included to stress the importance which should be attached to proper curing of the concrete pavement. Unless this importance is fully realized, efforts toward proper design will be wasted.

FORCES TO BE RESISTED BY THE PAVEMENT.

In the consideration of the various properties of concrete which affect its behavior in the pavement, brief mention has been made of several of the forces that the road slab must be designed to resist. It is necessary now to study these forces somewhat more in detail in order to properly appreciate their influence on the design itself.

Friction on the Subgrade.—Moisture and temperature changes cause corresponding variations in the length of the pavement slab. In order for the slab to vary in length, it must move over the subgrade. This movement will obviously be resisted by the friction developed between the pavement and the subgrade. It is highly important that something be known of the magnitude of these forces of friction and the effect on it of some of the more common variables met in concrete highway construction. A study of the forces required to move a concrete slab over various subgrade conditions was made some years ago by the U. S. Bureau of Public Roads.[8] These tests yielded valuable data on the magnitude of the friction co-efficient under the different test conditions. Figure 6 summarizes the results obtained. It was indicated that not only the type of subgrade material but its physical condition materially affected the co-efficient which, roughly, varied from 1 to 2. The higher figure is not a possible maximum, by any means, but may be considered a conservative value to use for design purposes. Laying concrete over badly rutted subgrades, over rock ledges, or over uneven old stone-road subgrades is bad practice as a consideration of the effect of increasing this friction on the subgrade will readily show.

Expansion and Contraction.—Because of the stresses developed by the constant effort of the concrete to drag itself over the subgrade against the forces of friction, so-called contraction cracks appear at intervals along the pavement. These cracks must be maintained, so generally some bituminous material is used as a filler with the unsightly result so common, giving an entirely erroneous impression as to the extent of the crack. If the contraction and expansion of the pavement can be controlled at joints, what an improvement would be made in the appearance of our concrete highways. Engineers, realizing this, have practically eliminated the longitudinal crack by the longitudinal center joint. Although the usual cause of longitudinal cracks is not contraction of the concrete, it illustrates a control of cracking by the use of constructed joints, which may be applied to the other dimension of the road slab. Expansion and contraction are, as has been pointed out, the result of changes in (a) moisture content, (b) temperature. While it is doubtful if these two agencies will ever produce their maximum effects at the same time, yet for design purposes it will be safer to assume that they do.

Let us examine the effect of each separately and see what their action will be. Moisture change affects concrete as it does wood. The change in

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[8] Friction Tests of Concrete on Various Sub-Bases, by A. T. Goldbeck. *Public Roads*, Vol. 5, No. 5, July, 1924.

length resulting from a given change in moisture condition varies considerably with the mix, size of specimen, and with the characteristics of the cement. But laboratory tests have indicated that concrete of a pavement mix may, under extreme conditions, contract 0.05 per cent upon drying out. This means a contraction of 0.6 in. in a 100-ft. road slab. As stated, this is an extreme case and probably will never be met in practice. However, considerable shrinkage does take place and it is safe to assume that part of it is permanent, as even under very wet subgrade conditions it is not likely that the original moisture condition is ever regained. Dr. Hatt has shown that concrete beams, exposed to weather conditions, after six months are contracted 0.03 per cent from their original length.[9]

Temperature is responsible for much of the stress in our concrete pavements. With a co-efficient which seems to depend on a number of factors but which for our purpose can be taken as 0.0000055, a contraction of 0.5 in. may occur in a 100-ft. length of pavement. The magnitude of this temperature effect will depend on the temperature conditions at the time the pavement was laid, but can easily reach the value given. If the pavement be laid in cold weather, very little contraction will be expected; but during the following spring, under combined hot weather and excessive moisture, expansion will take place which usually manifests itself by "blow-ups." Therefore, it would seem that proper design should provide for expansion joints to take care of at least the greater part of this movement.

As to the spacing of these joints, the problem is one of calculating the probable length of slab in which the forces of friction will develop to a value greater than the tensile strength of the cross-section, and then spacing the expansion joint slightly closer than the calculated strength. Assumptions will have to be made as to the co-efficient of friction and to the tensile strength of the concrete, which may properly be used. This method has been applied by A. T. Goldbeck,[10] who finds a probable length of 28.8 ft. in plain concrete slabs. This value corresponds closely with the average length of slab found by C. A. Hogentogler in the recent survey of concrete roads conducted by the National Research Council, during which cracking in over 2,000 miles of concrete pavement was closely observed.

Warping.—A difference in the moisture or temperature in the top and in the bottom of a pavement slab will result in a difference in length. Obviously, if there is a difference in length, there will be curvature. If there is curvature, there will be a variation in support from the subgrade. Research has again given us definite information. Over four years ago the Bureau of Public Roads made studies of warping which gave a very clear idea of the behavior of a road slab. During the day the edges of the pavement curled down, creating pressures as high as 6 lb. per sq. in. against the subgrade; during the night the corners came up something like 0.2 in.,

[9] The Effect of Moisture on Concrete, by W. K. Hatt. *Public Roads*, Vol. 6, No. 6, August, 1925.

[10] The Inter-relation of Longitudinal Steel and Transverse Cracks in Concrete Roads, by A. T. Goldbeck. *Public Roads*, Vol. 6, No. 6, August, 1925.

completely leaving the subgrade. Dr. Hatt, in the paper previously mentioned, observed an upward deflection of the corner of a slab of 0.2 in. due to moisture alone. This warping causes high stresses due, first, to the dead-weight of the slab itself, and second, to the unsupported condition of the edges under night traffic. The remedy is evidently to be found in a decrease of the size of the slab units of our pavement.

FORCES PRODUCED BY TRAFFIC.

So far we have considered only those forces which are exerted on the pavement due to climatological or other natural conditions. But the pave-

FIG. 7.—TRAFFIC CENSUS FOR CONNECTICUT.

ment is built to serve the needs of man and in so doing it is called upon to resist the action of traffic. In designing a pavement it is necessary to know or to predict what traffic it will probably have to bear. This is best accomplished through a traffic census.

The Traffic Census.—In order to determine the flow of traffic over a state or even a county highway system, numerous highway traffic studies have been made. From these the character, density and distribution of traffic in the various parts of a great commonwealth may be closely estimated. Fig. 7 shows such a traffic distribution map for the State of Connecticut. A study of such a map will show that the various arteries of the system bear considerably different amounts of traffic and that only

a very few of the highways will be called upon to carry the very heavy vehicles. This is most important from a design standpoint, as it clearly shows that not all pavements should be designed for the heaviest traffic loads but rather for the type of traffic loads to which it is most likely to be subjected during its normal life, as indicated by such a traffic census as has been described. This prediction properly should take into account a normal rate of development for the particular community served by the highway under consideration.

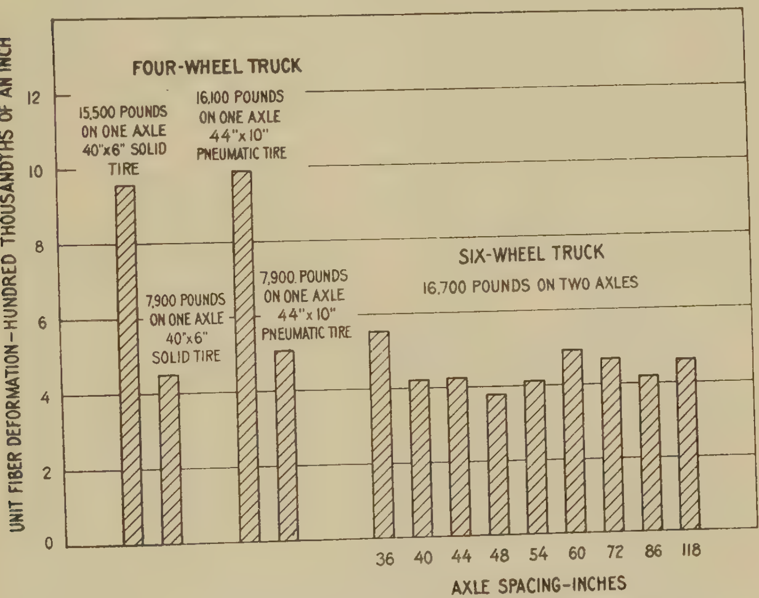


FIG. 8.—STRESSES PRODUCED BY 4-WHEEL AND 6-WHEEL TRUCKS.

The trend of motor vehicle design should also be duly recognized in making up this estimated maximum future wheel-load. For instance, the advent of the six-wheel or even the eight-wheel vehicle may have a very appreciable effect on pavement design. Recent investigations of the Bureau of Public Roads [11] have demonstrated that such equipment causes no more stress in the pavement than four-wheel vehicles of the same axle load, irrespective of the axle spacing. This will be seen by referring to Fig. 8. Certainly this provides a more efficient use of pavements and if this type of unit meets with public favor, it will assist the highway engineer in meeting the demands of increasing traffic with pavements whose cost is not excessive.

[11] The Six-Wheel Truck and the Pavement, by L. W. Teller. *Public Roads*, Vol. 6, No. 8, October, 1925.

Impact.—In speaking of wheel-loads we have so far considered only the actual dead weight of the truck wheel standing on the pavement. If a pavement were absolutely smooth, we could consider this as a maximum load condition, but such a road surface has not yet been built. Road roughness tends to produce varying wheel pressures or impact on the pavement. This impact effect is also a function of many other factors, such as tire equipment and speed. This subject of motor truck impact has received much study by the Bureau of Public Roads. It is very complex, but considerable information is now available concerning it. We know, for in-

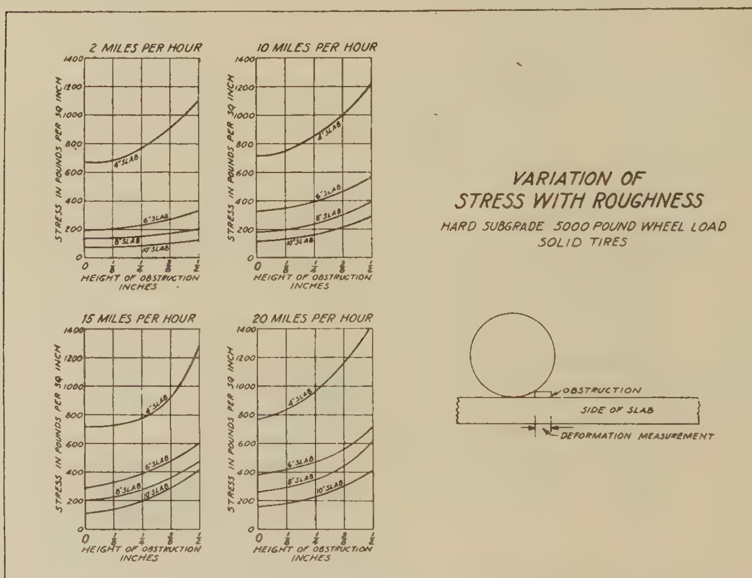
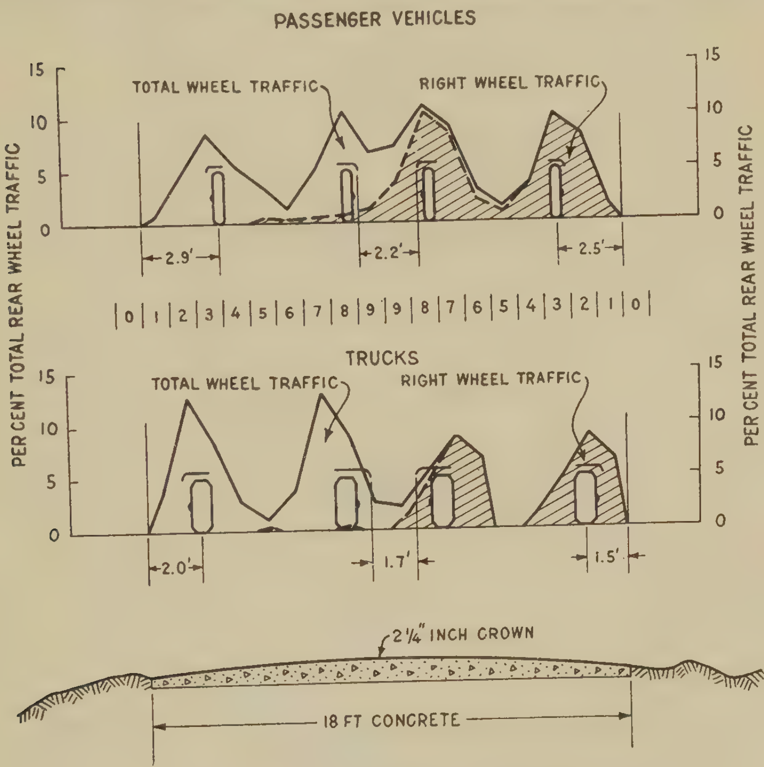


FIG. 9.—VARIATION OF FIBER STRESS WITH SURFACE ROUGHNESS.

stance, that a truck wheel traveling along an ordinarily smooth pavement at a speed of 12 to 15 miles per hour may readily produce wheel pressures against the pavement of several times the static magnitude. Fig. 9 shows how pavement stresses develop with road roughness on pavements of different thickness. Obviously, such impacts are very destructive to the pavement.

There are indications in research now being conducted that marked fatigue effects are present under impact load conditions and this adds to the potentialities of wheel impact, although to what extent we do not yet know. The researches mentioned may assist us in evaluating the effect of these impacts on concrete pavement in order that they may be properly taken care of in future design. However, it will always be necessary to

minimize these impact effects by building our pavements smooth in the first place. That the importance of this fact is recognized is evidenced by measurements of road roughness recently made in one of the federal-aid districts with an instrument designed and built by the Bureau of Public



TEST NO. 4 - TRANSVERSE DISTRIBUTION CURVES OF MOTOR VEHICLE TRAFFIC ON 18 FOOT CONCRETE PAVEMENT. SHOULDERS, IN FAIR CONDITION AND GRADE-LEVEL. DISTRIBUTION IS UNIFORM. AVERAGE SPEED 12 TO 25 MILES PER HOUR FOR TRUCKS AND PASSENGER VEHICLES RESPECTIVELY

FIG. 10.—LATERAL DISTRIBUTION OF TRAFFIC ON AN 18-FT. PAVEMENT.

Roads. These measurements show that since 1920 each year has shown an improvement in the smoothness of the surface of the concrete pavements laid, and that pavements laid during 1925 show only about one-half the surface roughness of those laid five years ago.

Transverse Distribution of Traffic on the Pavement.—Having studied the various factors which influence the magnitude of the traffic wheel loads,

it is now important to learn something of the probable position of these loads on the pavement. As a result of an investigation conducted by the Bureau of Public Roads,[12] there is some very valuable information on this subject. The lateral distribution is shown by this study to be influenced by such factors as grades, curves, crown, shoulders, traffic lines, etc., as well

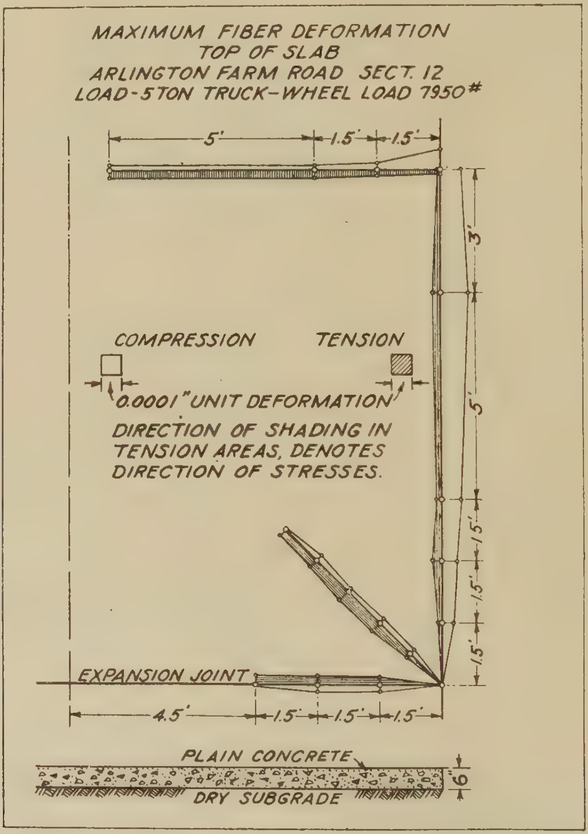


FIG. 11.—MAXIMUM FIBER DEFORMATION FOR ANY POSITION OF THE LOAD.

as by the type of traffic, but of greatest importance so far as pavement design is concerned, is the conclusion that, while heavy wheel-loads may pass over any point, the tendency is for this type of load to crowd the edge of the pavement. The significance of this will be touched on later. Fig. 10 shows a typical distribution chart for an 18-ft. concrete highway.

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[12] Transverse Distribution of Motor Vehicle Traffic on Paved Highways, by J. T. Pauls. *Public Roads,* Vol. 6, No. 1, March, 1925.

The conclusion to be drawn from the study of traffic loads is, then, that for maximum economy the cross-section should be designed for uniform strength.

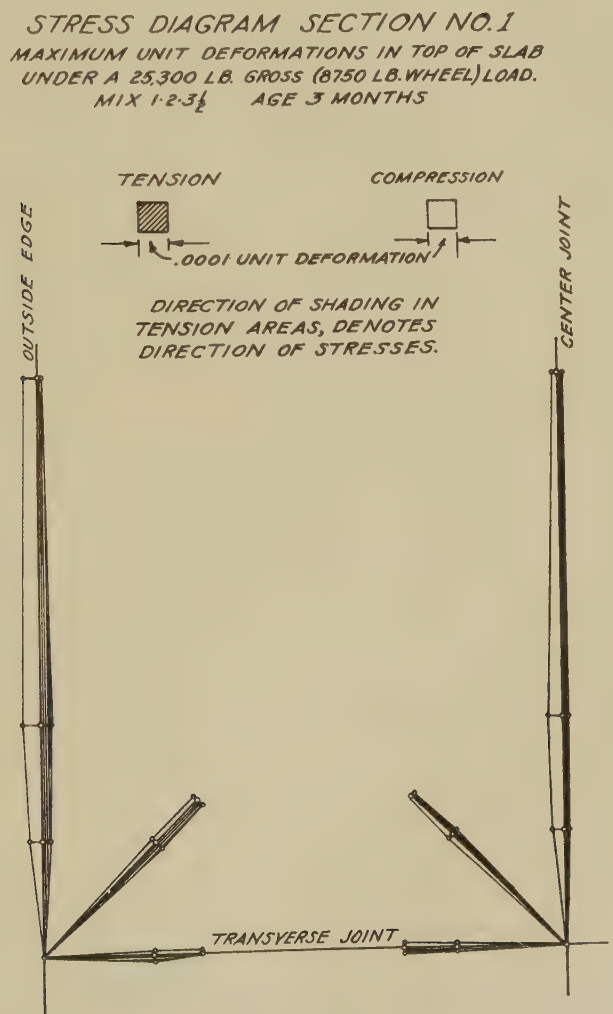


FIG. 12.—STRESS DIAGRAM FOR A THICKENED-EDGE PAVEMENT (COOK COUNTY).

DESIGN OF THE SLAB.

As a result of theoretical analysis, of stress measurements in various types of pavements, and of field investigations on the load-carrying ca-

pacity of different cross-section designs, the pavement of the thickened edge type has been shown to most nearly satisfy the requirement of uniform strength.

The Bates Road tests in Illinois some four years ago led to the adoption of the thickened edge design as the most economical for maximum strength. Another major investigation, the Test Road at Pittsburg, California, gave the cross-section of this type the highest rating as a result of their traffic load tests. At Arlington, Virginia, the Bureau of Public Roads determined the maximum deformations which could be produced at various points in concrete pavement slabs of uniform thickness for any position of the wheel-load. Some of these results are shown graphically in Fig. 11, and indicate very clearly that for a given load in a slab of uniform thickness the highest stresses obtainable are produced under a wheel when that wheel runs along the edge of the slab, indicating the edge of the slab as the weakest point. Further, they show that relatively high tensile stress occurs in the top of the slab when the wheel is on the corner of the slab. This stress, though high, is not so high as that along the edge, but is greater in magnitude than that caused by the same wheel load in the interior of this slab of uniform thickness.

More recently the Bureau of Public Roads has made stress measurements on pavements of the thickened edge design in the states of Illinois and Pennsylvania, and these indicate that uniform strength is obtained where the thickness of the central portion of the pavement is approximately seven-tenths of the edge thickness, providing that a longitudinal center joint of a type capable of transferring load is used. Otherwise there would have to be a provision for increasing the strength of the interior edge of the half slab. Fig. 12 shows a stress diagram obtained on one of these thickened-edge pavements.

Dr. H. M. Westergaard, in a report to the Committee on Structural Design of Roads of the National Research Council, December, 1925, presented a most interesting theoretical analysis of the stresses produced by a given load at three points of a concrete pavement of uniform thickness, the interior, the corner, and the edge.

To take one example from the tables he has calculated, it is indicated that in an 8-in. concrete pavement of uniform thickness, a wheel-load of 10,000 lb. will cause the following stresses, theoretically:

Load at	Resultant Fiber Stress
1. Interior	186 lb. per sq. in.
2. Corner	252 lb. per sq. in.
3. Edge	273 lb. per sq. in.

making certain assumptions as to the properties of the concrete, subgrade stiffness, and area of load application.

Thus do theory, experiment and practical road test all point to the thickened-edge type as the most economical cross-section.

The Corner Theory.—The researches just described have indicated that, all other conditions remaining constant, the highest stress which can be caused in a pavement of uniform thickness by a wheel-load is that directly under the load when that load is at the edge of the slab. When the wheel rests on the corner of the slab a stress, whose magnitude is somewhat less but still relatively high, occurs at a point somewhat removed from the position of the load. The exact calculation of these stresses is difficult and many assumptions are involved. An approximate solution of the one case where the load is applied at the corner has often been made by the application of the flexure formula to the corner of the slab, assuming it as a

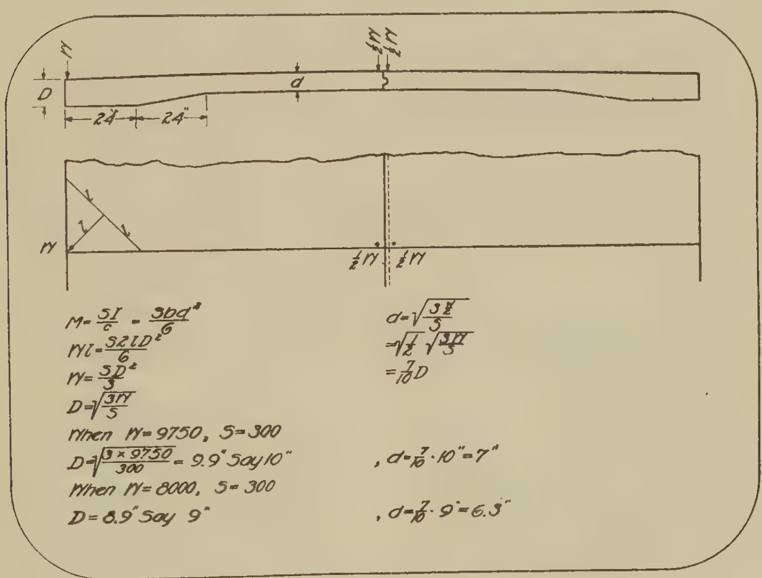
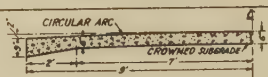
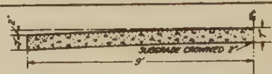
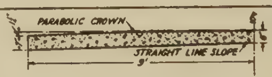
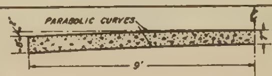
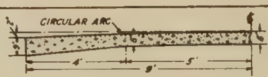
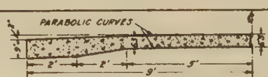
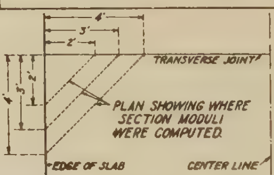


FIG. 13.—CORNER THEORY APPLIED TO THICKENED-EDGE SLAB.

cantilever without subgrade support. This solution is shown in Fig. 13 for both supported and unsupported corner conditions. It should be borne in mind that values arrived at by this solution are only approximate, because proper consideration is not given to a number of factors which would materially influence or modify the result. Among these may be mentioned impact, fatigue, subgrade support, cross-section and position of the load. Not all of these factors will have adverse effects. For instance, subgrade support is undoubtedly present to some extent in a majority of cases. Furthermore, it is unusual for a wheel-load to pass along the extreme edge of the pavement. It is probably due to such conditions that the corner theory has served its purpose so well.

The Section Modulus of Various Cross-Sections.—Design which does not result in economy has failed in its purpose. The object of the thickened edge design is to provide uniform resistance in all parts of the slab. Fig. 14 shows a number of cross-section designs for which the section

STATE	CROSS-SECTIONS.	SECTION MODULI						CONCRETE PER MILE	
		2'	%	3'	%	4'	%	CU. YDS.	%
ILLINOIS		339	100	459	100	572	100	1858	100
INDIANA		277	82	416	91	554	97	2053	110
NEW YORK		277	82	410	89	538	94	1906	103
NORTH CAR.		340	100	507	110	657	115	2154	116
PENNA.		390	115	545	119	678	118	1955	105
A.A.S.H.O.		458	135	659	143	823	144	2112	114



STUDY OF STRENGTH AND ECONOMY OF SEVERAL CROSS-SECTION DESIGNS.
U.S. BUREAU OF PUBLIC ROADS. DIVISION OF TESTS.

FIG. 14.—COMPARISON OF SECTION MODULI.

modulus has been calculated across the corner at distances approximating those at which a corner usually fails from traffic loads. As the quantity of concrete required for each design is also given, this furnishes an interesting comparison of the corner strength of the various sections with their cost per mile.

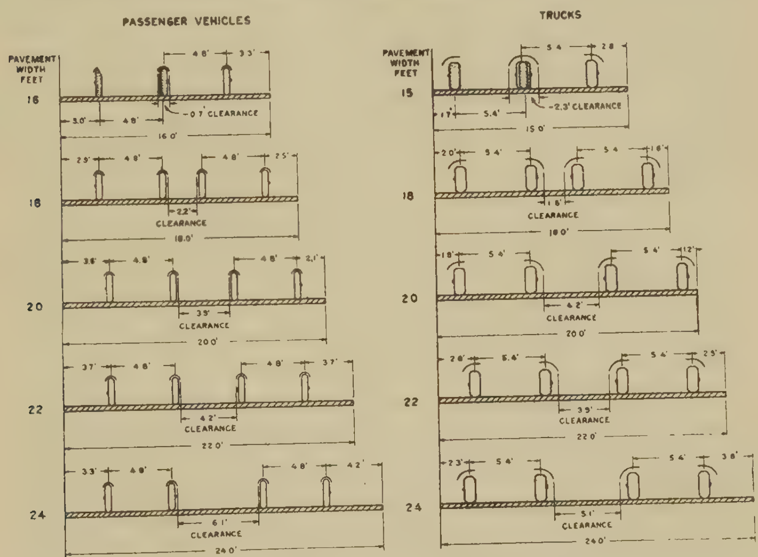
Width of Pavement Slab.—The survey of the transverse distribution of traffic previously referred to[13] furnishes information from which the designing engineer may determine the proper width of pavement for the needs of the predicted traffic conditions. Fig. 15 shows the average dis-

[13] Transverse Distribution of Motor Vehicle Traffic on Paved Highways, by J. T. Pauls. *Public Roads*, Vol. 6, No. 1, March, 1925.

tribution of traffic on pavements of various widths of from 15 to 24 ft. Probably the most important indication is that concrete pavements carrying two-way traffic should not have a width of less than 18 ft. Pavement widths of from 18 to 20 ft. seem to be ample for two-way traffic.

STEEL REINFORCEMENT.

A great amount of information has been obtained during the past year regarding the value of steel reinforcement in general and the adaptability of the various types to particular conditions. The most extensive study



THESE GRAPHS SHOW THE CLEARANCE OF PASSENGER VEHICLES AND TRUCKS ON WIDTHS OF PAVEMENT VARYING FROM 15 TO 24 FEET. IT IS TO BE NOTED THAT 18 FEET IS THE MINIMUM WIDTH TO GIVE SUFFICIENT CLEARANCE FOR ALL VEHICLES. THE CLEARANCE INCREASES IN GENERAL WITH THE WIDTH OF THE PAVEMENT AND BECOMES GREATEST WITH THE 24 FOOT PAVEMENTS AND EVEN ON THESE THERE IS NOT ENOUGH ROOM FOR A THIRD LINE OF TRAFFIC. IT SEEMS PROBABLE FROM THESE DATA THAT ANY WIDTH ON TANGENTS GREATER THAN 20 FEET IS EXCESSIVE FOR TWO-WAY TRAFFIC. THE TRUCKS TRAVEL CLOSER TO THE EDGE THAN THE PASSENGER VEHICLES AND FOR THIS REASON AN 18 FOOT PAVEMENT PROVIDES ABOUT THE SAME CLEARANCE FOR BOTH TYPES OF VEHICLES. IT IS PROBABLE THAT A TRAFFIC CENTER LINE PROPERLY POLICED WOULD SEPARATE AND HOLD PASSENGER VEHICLES TO THE PROPER SIDE OF THE ROAD

FIG. 15.—DISTRIBUTION OF TRAFFIC ON PAVEMENTS OF VARIOUS WIDTHS.

of this problem is the field survey conducted by the Highway Board of the National Research Council under the direction of C. A. Hogentogler. In this investigation comparison was made between plain and reinforced-concrete pavements and also between those in which various types of steel reinforcement were used.

The Bureau of Public Roads has made a continuous study of a special test pavement laid on the Columbia Pike, near Washington, D. C. This test road, which is approximately two miles in length, contains 32 experimental sections varying such features as thickness, reinforcement, and cross-section of slab. State highway departments have also been active in attacking this problem and the result of all of these efforts has been that now, for

the first time, steel can be used in the pavement design with some confidence that the results expected will be attained.

In the past reinforcement has, in general, been added to the pavement as an added precaution against some unfavorable condition, such as bad subgrade, present or anticipated heavy traffic, to strengthen edges or corners or to reduce cracking.

Research has shown that steel cannot be economically used to increase the structural strength of pavement slabs. This was indicated in tests conducted by the Bureau of Public Roads several years ago,[14] and has been substantiated by other investigators. The use of excessive amounts of steel, in addition to being costly, presents other difficulties.

The field studies mentioned have indicated that steel reinforcement, if properly used, will not only not reduce cracking but may actually produce more cracking than would occur in a plain concrete pavement. The most general case of this kind found was that in which longitudinal steel was used in continuous bond for long distances, particularly where the bars were heavily grouped along the edges of the pavement. Under such conditions edge and corner cracks developed and, where the tensile strength of the steel exceeded that of the concrete cross-section, additional transverse cracking developed.

Considering this observed behavior from the standpoint of theory, it would seem that there are very good reasons for at least some of these results.

In the first place reinforcement cannot economically be used to prevent cracking in the pavement due to stress set up by temperature or by traffic. Steel does not function economically in any reinforced structure until cracking does take place. This is shown in the following table, which was calculated by A. T. Goldbeck in a paper previously mentioned.[15]

TABLE SHOWING EFFECT OF LONGITUDINAL STEEL ON TRANSVERSE CRACKING.

Amount of longitudinal steel	Required spacing of transverse joints	
	(1) For no intermediate cracks Feet	(2) For no wide intermediate cracks Feet
Plain concrete	28.8	28.8
4¾ in. round bars (a = 1.76 sq. in.)	29.2	32.6
8¾ in. round bars (a = 3.52 sq. in)	29.6	65.2
12¾ in. round bars (a = 5.28 sq. in.)	30.0	97.8

Increased transverse cracking is to be expected where heavy steel reinforcement is used with continuous bond for long distances. The reason for this is that when the transverse crack occurs in a plain concrete slab the

[14] Impact Tests on Concrete Pavement Slabs, by L. W. Teller. *Public Roads*, Vol. 5, No. 2, April, 1924.

[15] The Inter-Relation of Longitudinal Steel and Transverse Cracks in Concrete Roads, by A. T. Goldbeck. *Public Roads*, Vol. 6, No. 6, August, 1925.

concrete on either side of the crack is immediately relieved of all stress. In the case of the reinforced pavement the steel does not crack with the concrete and transmits considerable tension across the transverse crack into the adjoining slab. This tension shortens the length of that slab to a degree corresponding to the stress transferred. This effect was first remarked in the study of the experimental sections on the Columbia Pike by the Bureau of Public Roads.[16]

Heavy grouping of longitudinal steel along the edges of the pavement, when used in continuous bond for long distances, will cause edge cracks and corner cracks. Here, again, the cause is the transfer of stress across a crack by the steel. The rate at which this stress is taken up depends on the bond strength of the concrete. If the bond strength accumulates faster than the tensile strength of the concrete across the section involved, a failure in tension will occur. On the other hand, if the tensile strength exceeds the bond developed, a bond slip will take place near the transverse joint. These effects have all been observed in the study of the experimental sections of the Columbia Pike.

It is apparent, then, that where steel is used in continuous bond for long distances the following factors contribute toward the formation of corner breaks:

(a) Reinforcement concentrated near the edge of the pavement.
(b) Steel highly deformed so as to produce unusual bond strength.
(c) Weak concrete cross-section (such as thin-edge pavements).

By assuming reasonable values for the tensile strength of concrete and for the bond strength of steel and by making certain reasonable assumptions as to the effective cross-section of concrete which resists corner breaking, it would appear that from the method shown in Fig. 16:

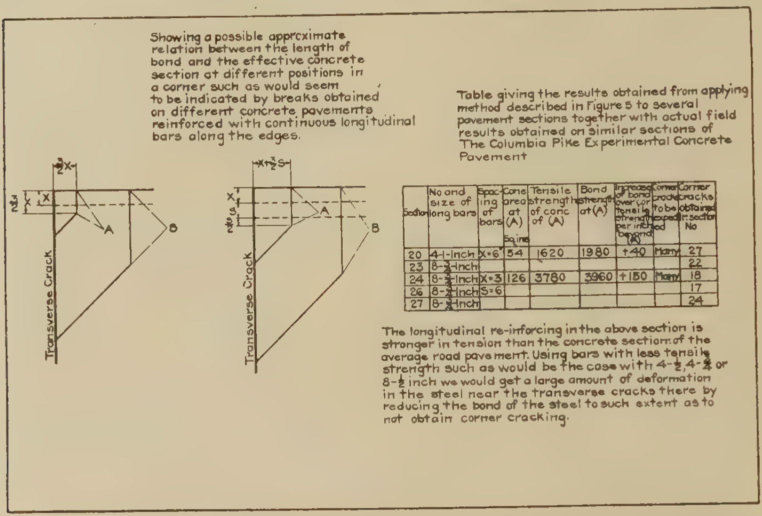
1. One ¾-in. normally deformed bar, placed not less than 6 in. from the edge approaches the maximum condition in which a single bar can be used without corner cracks.
2. Two ½-in. normally deformed bars placed 6 in. and 12 in. from the edge approach the limiting condition when two bars can be used without corner cracking.

This theory regarding corner cracking has been developed as a result of studies of reinforced pavements, the details of whose construction were definitely known. An example of the type of cracking to which this theory refers is shown in Fig. 17.

We have seen what is to be expected theoretically from the use of steel reinforcement placed in continuous bond for long distances. Fig. 18 presents in chart form the condition, with respect to cracking, of the various reinforced sections in the Columbia Pike. The effect of large size longi-

[16] Reinforcing and the Subgrade as Factors in the Design of Concrete Pavements, by J. T. Pauls. *Public Roads*, Vol. 5, No. 8, October, 1924.

tudinal steel is noticeable, both on corner cracks and transverse cracks. It will also be observed that transverse steel as used in these sections has a marked effect on the amount of transverse cracking. Fig. 19 shows the



Section	No and size of long bars	Spacing of bars	Conc area at (A) Sq ins	Tensile strength of conc of (A)	Bond strength at (A)	Increase of bond over corr tensile strength per inch beyond (A)	Corner cracks to be expected	Corner cracks obtained in section No
20	4-1-Inch	X=6	54	1620	1980	+40	Many	27
23	8-½-Inch							22
24	8-½-Inch	X=3	126	3780	3960	+150	Many	18
26	8-½-Inch	S=6						17
27	8-½-Inch							24

FIG. 16.—THEORY OF CORNER BREAKS DUE TO STEEL REINFORCEMENT.

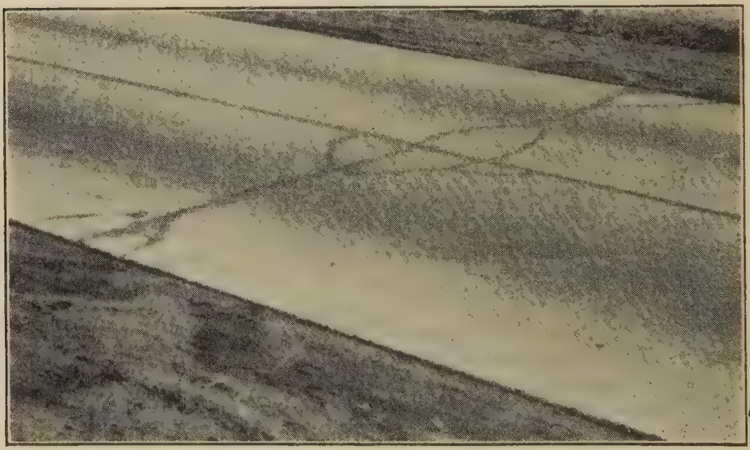


FIG. 17.—PHOTO OF CRACKING IN COLUMBIA PIKE, ILLUSTRATING THE THEORY.

present appearance of one of these sections where heavy transverse reinforcement was used. The value of steel bars is not qustioned, but it does seem that the manner in which they have been used is open to criticism.

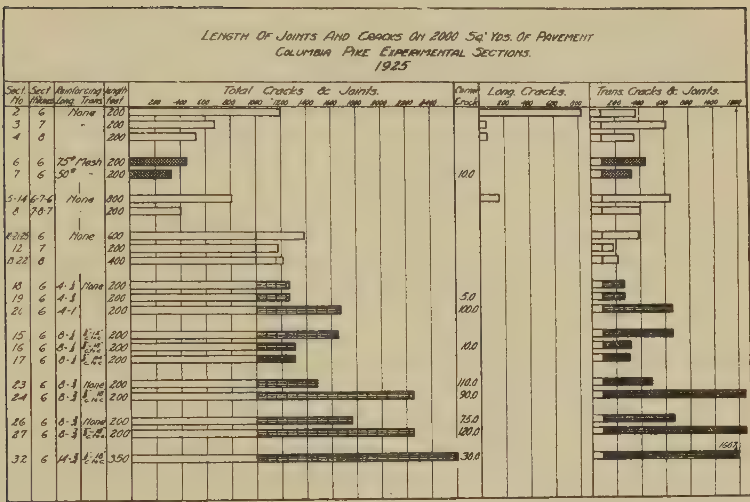


FIG. 18.—CHART OF THE CRACKING IN COLUMBIA PIKE.

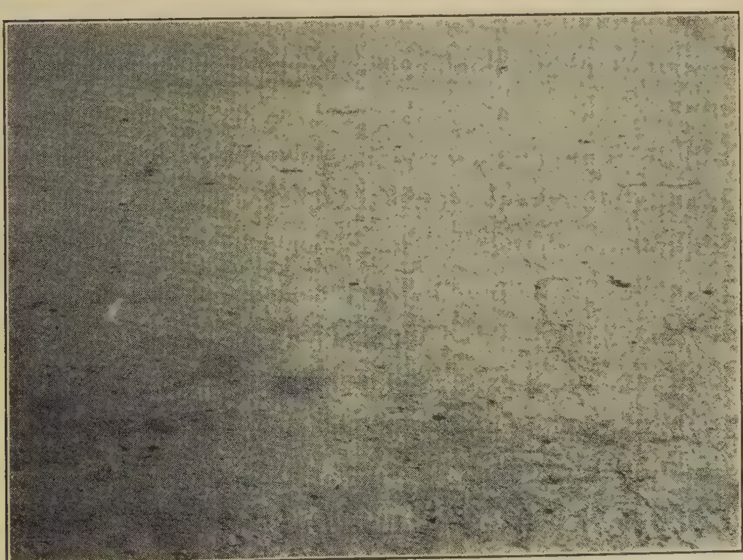


FIG. 19.—PHOTO OF TRANSVERSE CRACKING CAUSED BY TRANSVERSE STEEL.

A few of the most important indications of the recent investigations and field condition surveys, previously mentioned, may be stated as follows:

1. The impracticability of having longitudinal steel in bond function over considerable lengths was evidenced, both in the case of bars and mesh, by increased transverse cracking.
2. Light mesh reinforcement seemed to be most effective in reducing cracking.
3. Large transverse bars induced transverse cracking of the pavement over the bars.
4. Concentrating the steel along the edges of the pavement, when the bond was continuous for long distances, produced corner cracking.

It is believed that steel reinforcement can be used in concrete pavements with beneficial results, providing it is properly designed.

Proper design is considered to be of such a nature that the steel will be used only for the purpose of providing resistance to those agencies against which it has been found to be effective. Further, proper design should provide against the use of steel in any manner which is known to produce adverse effects.

To make the design most effective, the following precautions are considered of great importance:

(a) Longitudinal steel should not be used in lengths exceeding 30 to 40 ft. where the bond is continuous.
(b) Good distribution of the steel should be provided through the use of smaller rods more closely spaced.

Steel Dowels.—It is believed that one of the most important uses to which steel can be put in a concrete pavement is as a dowel across a constructed joint. For this purpose, across transverse joints the steel should not be in bond with the concrete. From elastic curves obtained under wheel-loads it is indicated that it is desirable for dowel-bars to be rather closely spaced. For instance, practically all of the deflection of a 6-in. slab acted on by a loaded 5-ton truck takes place over a length of 8 ft. If dowel-bars along a longitudinal joint are to assist in transferring load across the joint, it would seem that in the case mentioned they should be spaced at intervals of not more than one-half this distance, or approximately 4 ft. along the center-line of the pavement.

CONCLUSION.

In order to secure a perspective on the subject of pavement design, the conclusion of this discussion will be devoted to a very brief review of the most important points which have been mentioned.

The subgrade exerts a marked influence on the behavior of the pavement throughout its life. As the physical condition of the subgrade is dependent on the amount of moisture in it, the subgrade problem is largely

one of internal and external drainage and must be solved by the improvement of these conditions. Certain soils, particularly those of high clay content, make very unstable subgrades in the presence of excessive moisture. These materials must be drained or their structure so altered that they will lose their instability under the maximum moisture conditions obtaining. In general, any treatment which would stabilize an earth road will improve a subgrade.

Due to temperature and moisture variation, the phenomena of expansion, contraction and warping tend to begin as soon as the concrete is set up. These must be reduced to a minimum by proper curing in order that the resistance of the concrete to bending and direct tension may develop.

A prediction as to the probable maximum wheel-load to be used in the design can best be made by reference to traffic census studies, with due consideration to future development of the community to be served. The trend of motor vehicle design should also be considered in making this estimate. In addition to the dead-weight of the wheel on the pavement, account must be taken of the effect of impact. This impact must be reduced to a minimum through the construction of smooth pavement surfaces. The maximum wheel-load may be applied at any point on the pavement so that, for maximum economy, the road slab should be designed for uniform strength.

Practical field tests, research experiments, and theoretical analysis all point to the thickened-edge design as most nearly satisfying the requirement for a design of uniform strength. Traffic surveys indicate that 18 ft. is the minimum width of pavement which will safely accommodate two-way traffic. In order that the pavement may secure more uniform support from the subgrade, it should be subdivided by constructed joints.

Steel reinforcement, when properly used, is beneficial to a concrete pavement. When improperly used, it will produce more cracking than will be found in a plain concrete pavement under the same conditions of support and of load. The use of steel in continuous bond for long distance is not considered good practice, nor is the concentration of steel reinforcement along the edges of the pavement slab. For a given amount of steel, better pavements will be obtained if the steel is distributed by using smaller rods more closely spaced. Steel reinforcement should not be used with the idea of preventing cracks in concrete pavements, but rather for the purpose of so controlling the cracking which will occur that structural strength will be maintained and pavement failure will be prevented.

DISCUSSION.

Mr. Johnson.

T. H. Johnson (*By Letter*).—Radical as I know they will seem, the convictions of mine herein stated are the result of fourteen years of experience, covering responsible charge, from writing specifications to the acceptance of the work, of what would be the equivalent of around 300 miles of 18-ft. highway pavement. This experience has covered an area of 500 miles east and west by 200 miles north and south and including portions of four states, Iowa, Minnesota, South Dakota and Nebraska. It has covered every possible type of soil and the most trying climatic conditions.

It is my belief that a fuller perception of the possibilities that lie in the direction indicated by these observations will hasten progress toward a rational practice in building concrete pavements, and will persuade men that the final solution of the paving problem is not to be found, primarily, in structural design.

Concrete *pavements* are not designed. Rather, pavement *slabs* are designed, and then the constructor *imparts* to the slab such of the essential qualities of pavement as his knowledge and skill permit. A pavement is in reality a work of art, and is not primarily amenable to the rules of design. It is the accomplishment of a skilled hand guided by trained, experienced intelligence. Its characteristic qualities are *imparted* qualities, qualities that are not *inherent* in the material of which it is made, or, to any controlling degree, in the form that may be given them.

There is no such thing possible as a correct estimate of all the forces that will act on the pavement, consequently no such possibility as designing a structure that will resist all of them. Nor is there any such thing as anticipating exactly the values, as affecting the quality of the pavement, of a great variety of constantly changing conditions surrounding the laying of the pavement, each of which, in its turn, has its influence upon the quality, and making provision for them in the design.

Structural design will help, but the main dependence for life and usefulness of the pavement, must be upon qualities *imparted* to the substance of the slab during the process of laying and finishing. A very clear line of demarcation should be had between the *making of the concrete*, and the *laying of the pavement*, because they are two entirely separate operations.

It is during the latter operation, after the concrete has been placed on the subgrade, that the qualities of hardness, toughness, density, tensile and compressive strength, impermeability, resistance to abrasive wear, resistance to the effects of changes in temperature and moisture, resistance to "checking," cracking, etc., must chiefly be imparted. The impartation of these qualities lies almost wholly beyond the domain of structural design, and largely beyond the scope of standard practice.

It is correct that the influence of the subgrade is one of the great problems to be solved, but it is not so much because of "the way it affects the behavior of the pavement *throughout* its life" as it is the influence it has at the time of its *borning*, on its ultimate quality. Its most harmful effect, so to speak, is due to the fact that it is not permitted to function in its normal way at the time the pavement is being laid, in the absorption of a portion of the excess water necessarily put into the mix to secure workability, *after the need of workability has passed.*

The authors say it is important to have the subgrade thoroughly moist at the time the concrete is placed. If this is correct, Prof. Abrams is wrong. He says, "the importance of any method of mixing, handling, placing and *finishing* concrete which enables the builder to reduce the water content to a minimum, is at once apparent." If 10 to 15 per cent excess water is added to secure workability, it means an addition of five to seven pints per square yard of the 6-in. pavement, and it is hardly conceivable that the absorption of say half that quantity of water from a square yard of slab by an equal area of absorptive base, would occasion a material increase in the volume of the base. It would, however, according to Prof. Abrams, mean much in the added strength of the concrete.

Neither this absorption by the base, nor the evaporation from the surface is the cause of the surface checking. Checking is due to the more rapid removal of the excess water from the surface than from other portions of the slab. If the base is allowed to absorb a portion of the excess water from the slab, and the remainder is afterward drawn by adequate means to the surface and evaporated by constant manipulation, the surface will at all times be the wettest portion of the slab, and consequently will not crack. This method of drying out the slab from the bottom upwards, operates to prevent checking, even on the loess soils of Iowa.

This early and systematic removal of the excess water hastens the development of the tensile strength, and enables the pavement to resist the stresses due to setting. This method of finishing renders the slab less susceptible to changes of temperature and moisture, produces a denser slab with a surface that is hard, dense, impervious and highly-resistant to abrasive wear.

The discovery of the real importance of the water content of the mixed concrete, as expressed by the water-cement ratio, seems to have been accepted as settling to a finality the matter of *quality* in *pavement*, but it will eventually be found that removal of the excess water necessary for workability, *after this need has passed*, and the subsequent manipulation of the surface to close up the voids from which the water has been removed, will do for concrete *pavements* what Prof. Abrams has done for *concrete*, and will overcome many difficulties now considered beyond reach except through some extraordinary and as yet undetermined treatment of the subgrade.

Outline of Tests on 300-Ft. Reinforced-Concrete Chimney.

By Benjamin Wilk.*

At the 1925 convention of the American Concrete Institute, E. A. Dockstader of the chimney committee, made a report on tests on a chimney at New Bedford, Mass., to determine the temperatures in reinforced-concrete chimney shells. On account of the lack of available data, Mr. Dockstader suggested that similar tests be made on other chimneys to accumulate data from which positive conclusions might be drawn. This suggestion led the Universal Portland Cement Co. to co-operate with the chimney committee in making tests on a 300-ft. reinforced-concrete chimney, then being designed for its Duluth plant.

The chimney is 300 ft. high, 24 ft. inside diameter at the base and 18 ft. inside diameter at the top. The thickness of wall at the base, which is also the ground line, is 36 in. tapering to 6 in. at the top. An inside concrete lining 6 in. thick starts 27 ft. above the base and extends to 145 ft. above the base. The air space between the outside wall and the lining is 6 in. The stack is supported on an octagonal base 8 ft. thick and 66 ft. in longest dimension. Design of the stack is shown in Fig. 1. The chimney was designed by the engineering department of the Universal Portland Cement Co., and constructed by the Weber Chimney Co. of Chicago.

A sub-committee of the chimney committee was formed to act as an advisory committee on these tests. As the chimney was not under construction when the committee had its first meeting, a good opportunity presented itself for outlining a series of tests that would be comprehensive and which would develop as much information as possible on the effect not only of heat but also of wind on chimneys. The committee outlined the tests to determine:

(1) Temperature gradient through the lining and wall of a reinforced-concrete chimney.

(2) Nature and intensity of stresses developed in concrete and steel reinforcing due to wind pressure upon the chimney and as a result of flow of heat through the wall.

(3) Wind velocity adjacent to the chimney.

(4) Distribution of the pressure developed on the cylindrical surface of a chimney by wind.

(5) The nature and extent of the deflection of the chimney due to wind pressure and heat.

*Assistant Western Manager, Service Bureau, Universal Portland Cement Co.

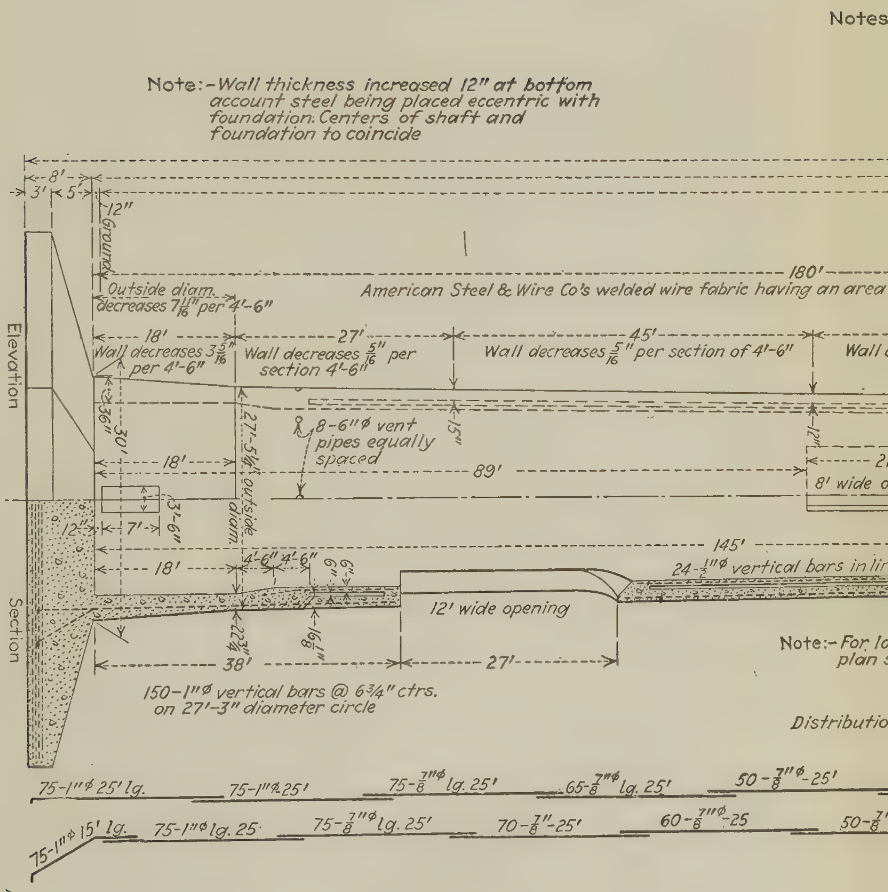
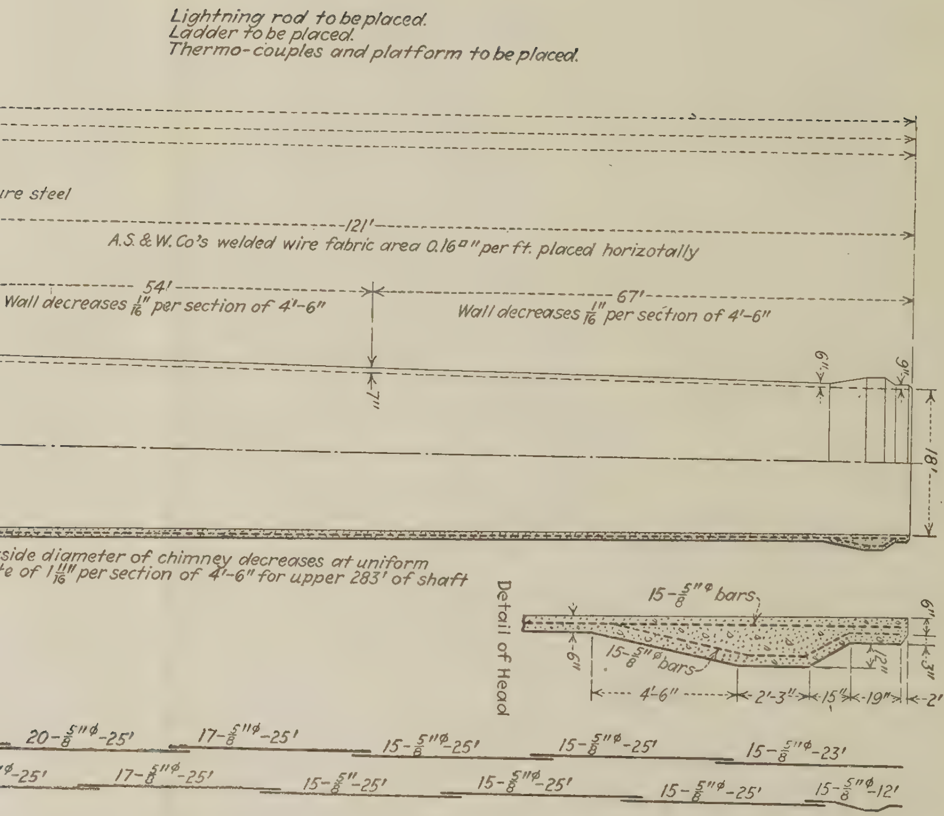


FIG. 1.—DETAILS OF TEST CHIMNEY.


Lightning rod to be placed.
Ladder to be placed.
Thermo-couples and platform to be placed.
ire steel
121'
A.S. & W. Co's welded wire fabric area 0.16□" per ft. placed horizotally
54'
Wall decreases 1/16" per section of 4'-6"
67'
Wall decreases 1/16" per section of 4'-6"
7"
6"
9"
18'
side diameter of chimney decreases at uniform
te of 1 11/16" per section of 4'-6" for upper 283' of shaft
Detail of Head
15-5/8"φ bars
15-5/8"φ bars
6"
6"
3"
12"
4'-6"
2'-3"
15"
19"
2'
20-5/8"φ-25'
17-5/8"φ-25'
15-5/8"φ-25'
15-5/8"φ-25'
15-5/8"φ-23'
"φ-25'
17-5/8"φ-25'
15-5/8"-25'
15-5/8"φ-25'
15-5/8"φ-25'
15-5/8"φ-12'

TEMPERATURE GRADIENT THROUGH CHIMNEY WALL.

In studying temperature gradient through the lining and wall of the chimney it was decided to use thermo-couples and to locate them at five heights or levels as follows:

(a) Just below the lower breeching.
(b) Just below the upper breeching.
(c) Just below the top of the lining.
(d) Just above the top of the lining.
(e) Near the top of the chimney.

TABLE I.—POSITION OF THERMO-COUPLES IN 300-FT. CONCRETE CHIMNEY.

Switch Point	Thermo-Couple Number	Location	Elevation
1	Z	Outside	92.5
2	B	2″ in	
3	A	2″ from inside face	
4	Z^2	Outside	159.5
5	B^2	2″ in	
6	K	⅓ T	
7	G	⅔ T	
8	F	2″ from inside face	
9	P	CL of air space	
10	T	CL of lining	
11	V	1″ in stream	
12	Z^3	Outside	204.5
13	B^3	2″ in	
14	K^2	½ T	
15	G^2	2″ from inside face	
16	Q	CL of air space	
17	U	CL of lining	
18	W	1″ in stream	
19	Z^4	Outside	231.5
20	B^4	2″ in	
21	M	½ T	
22	J	2″ from inside face	
23	R	1″ in stream	
24	XA	¼ D	
25	XB	½ D	
26	Z^5	Outside	362.0
27	B^5	2″ in	
28	N	½ T	
29	M^2	2″from inside face	
30	S	1″ in stream	

Top of lining 220 feet.

Elevation of base of chimney (ground line) 75.0.

In order to determine the variation of temperature of the temperature gradient at any particular level, it was decided to place the thermo-couples:

(a) In the air adjacent to the exterior surface of the wall.
(b) At several intervals in the chimney wall according to the thickness of the wall.
(c) In the air space between the wall and the lining.
(d) In the center of the lining.
(e) In the gas stream adjacent to the interior surface of the concrete.
(f) In the middle of the gas stream.

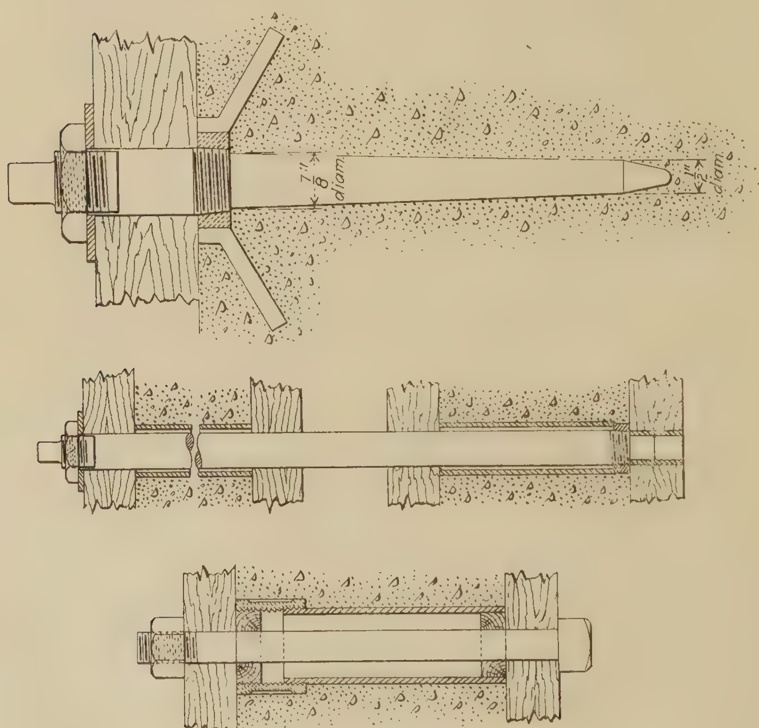


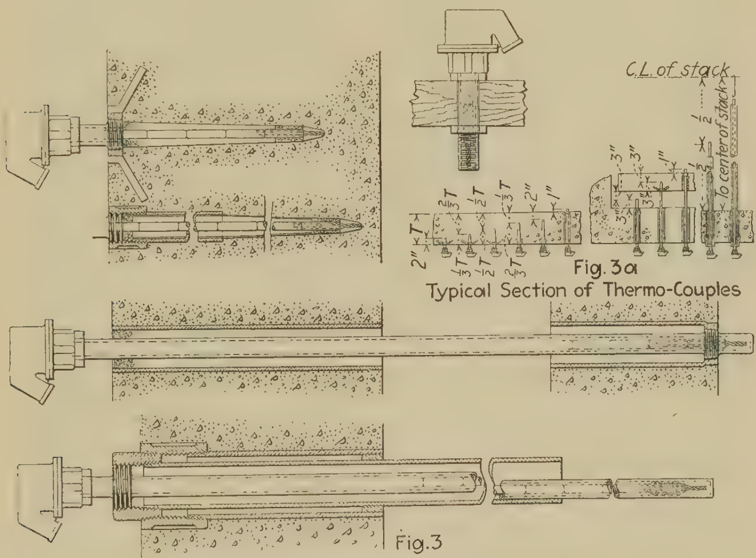
FIG. 2.—DESIGN OF MANDRELS.

A total of 30 thermo-couples has been placed in the chimney. Table I shows the position of the thermo-couples and refers to switch points.

Iron constantan thermo-couples were used. The thermo-couples in the concrete wall are of the wire type and are directly in contact with the concrete, but the thermo-couples exposed to the air and to the gas stream are of the pipe type so as not to be affected by moisture in the air or by

condensation of the gases. In general the space occupied by the thermo-couple was formed by a solid steel mandrel. The mandrel carried a permanent insert consisting of a standard brass locknut, and an outside nut which clamped against the form and held the mandrel in position. The mandrel was stripped by turning the squarehead which projected outside of the form until the mandrel was free of the locknut. Fig. 2 shows the design of the mandrels.

Threaded plugs were fabricated on the thermo-couple stems which screwed into permanent inserts at the face of the chimney, holding thermo-couples in a rigid position. In inserting the thermo-couples a packing of



FIGS. 3 AND 3A.—METHOD OF INSERTING THERMO-COUPLES.

asbestos was placed between the spagetti around the thermo-couple wires and the concrete so as to make a watertight job. Fig. 3 shows method of inserting thermo-couples. Fig. 3a is a typical section. Where the thermo-couples extend into the inner lining or far into the gas stream, a pipe sleeve was left in the chimney wall to aid in removing the mandrel.

Thermo-couple readings are taken at the base of the chimney through three 10-point switches read on a Leeds & Northrup automatic cold junction compensating potentiometer with a scale reading directly from minus 50 deg. F. to plus 600 deg. F. The readings already taken show that the temperatures are quickly read directly on the dial of the potentiometer. The thermo-couples have been calibrated to insure their accuracy.

STRAIN GAGE READINGS.

In determining the nature and intensity of stresses developed in the concrete and reinforcing steel as a result of flow of heat through the wall, and due to wind pressures on the chimney, gage points for Berry strain gage reading have been set at the eighth points around the chimney 16 ft. above the base and at three locations alongside of the chimney ladder at 90 ft., 135 ft. and 162 ft. above the base. Each set of gage points consists of three points on two adjacent horizontal rods and also three points on two adjacent vertical rods. This makes it possible to get a good check on individual readings at any particular point. The length between gage points is approximately 8 in. The Berry strain gage can register a move-

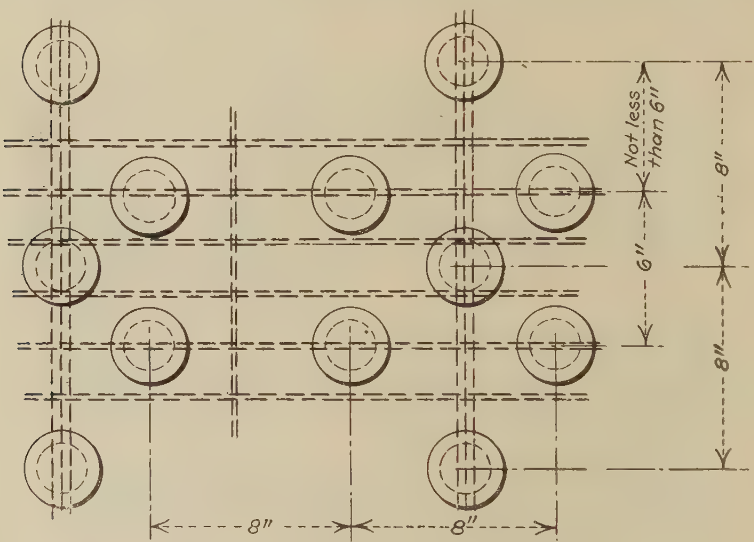


FIG. 4.—SET OF STRAIN-GAGE POINTS.

ment of 1/50,000 of an inch in a length of 8 in. or 1/400,000 in. per inch. Professor Lagaard of the University of Minnesota who is widely experienced in strain gage readings, is supervising this phase of the tests. Fig. 4 shows a set of strain gage points.

It was the idea of the committee to take strain gage readings:

(a) Before the chimney was heated to determine the stresses while the chimney was free from internal stresses caused by the heat.

(b) While the chimney would be heated to gain information on how the stresses developed as the heat rises.

(c) At stated intervals after a constant heat condition exists to determine the effect of both the internal and external forces on the chimney. These measurements would also be taken during times of high wind.

It is very possible that through the co-operation of the University of Minnesota the committee will have the use of four McMillan strainagraphs to record movement in the steel and concrete. McMillan strainagraphs have been used very successfully in recording strains on ships, bridges and buildings. It is expected that the four strainagraphs will be used on the sets of strain gage points near the base of the chimney.

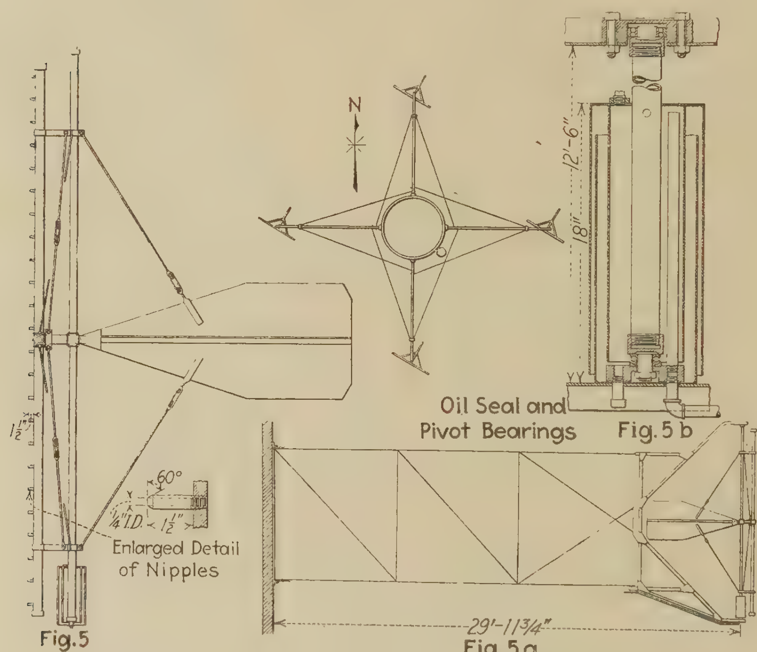
Wind Velocity.

In determining wind velocity adjacent to the chimney, considerable work was done in developing a method which would be accurate. Wind velocity readings taken with U. S. Weather Bureau anemometers which are of the revolving cup type, do not show violent gusts of wind which may last for only a few seconds. These gusts are of extreme importance and must be studied. The Burton anemometer developed by C. O. Burton of the Minnesota Steel Co., Duluth, Minnesota, is also of the revolving cup type but it only registers wind at a definite point. It however shows gusts and has registered wind velocities at Duluth as high as 130 miles per hour. It was the intention of the committee at first to use flat plates at various points around the chimney which would register wind pressure through Pitot tubes read at the base of the chimney. Through the efforts of Professor Maurer of the chimney committee, the co-operation of E. N. Fales, aero engineer of the U. S. Army Air Service, McCook Field, Dayton, Ohio, was obtained. Mr. Fales has given a great deal of study to the subject of wind velocities and wind pressures. He suggested the use of integrating impact pressure tubes for measuring wind velocities as well as for measuring wind pressures. This method has been used successfully in aeroplane work and has to a large extent been developed in wind tunnel tests at Dayton, Ohio. It has been found that the wind velocity and wind pressure vary considerably over a comparatively small area. For that reason small flat plates are not considered accurate. The integrating impact pressure tubes will therefore give much more accurate information as to the velocities and pressures to which the chimney is subjected than the plates originally considered.

In following Mr. Fales' suggestion, two integrating impact pressure tubes 12 ft. long at right angles to each other, forming a cross, will be set 30 ft. out from the chimney at the quarter points around the chimney. The horizontal 12-ft. integrating tube will be 234 ft. above the base of the chimney. In effect, these two 12-ft. tubes will register the average wind velocity over an area of 144 sq. ft. The tubes will be 1½ in. in diameter with holes on one side spaced 6 in. apart. As the registering of the wind pressure is effected by the curvature of the tubes, ¼-in. nipples have been inserted into the holes so that the ends of the nipples will always be at right angles to the direction of the wind. The tubes will be supported on steel outriggers and so pivoted and controlled by a weather vane that the

tubes will always be in the face of the wind. Fig. 5 shows a detailed design of the 12-ft. integrating tubes; Fig. 5a shows the design of the supporting outriggers. The pressure exerted by the wind will be transmitted to the base of the chimney by means of a liquid seal and ⅜-in. pipe leading from the seal to the base of the chimney. Fig. 5b shows the design of the liquid seal.

The integrating impact pressure tubes and the outriggers are now being fabricated and will be raised into position about May 1, 1926.



FIGS. 5, 5A AND 5B.—INTEGRATING TUBES, OUTRIGGER AND LIQUID SEAL.

WIND PRESSURE.

In determining the distribution of the pressure developed on the cylindrical surface of the chimney by wind forces, it was decided that pressures should be secured for two distinct reasons:

(a) To determine the nature of the action of wind on a cylindrical surface. Tests on small models show that a positive pressure is exerted on only a small proportion of the cylindrical surface, approximately 70 deg., while a negative pressure or partial vacuum exists on the remainder of the circumference. This theory is not as yet used in the design of towers and chimneys, but it is agreed that definite information on this subject on a

chimney 300 ft. high would show the relation between coefficients as determined on a small model and on a large structure, and the data would be a distinct addition to scientific knowledge.

(b) Because data on wind and pressure as secured coincident to data on the stresses on concrete and steel reinforcing would effect a check on formulas for calculating these stresses.

The integrating impact pressure tube idea as used in determining wind velocity will also be used in obtaining wind pressures on the chimney. One-inch integrating impact pressure tubes 3 ft. 2 in. long, have been placed in the face of the chimney at 18 equidistant points around the chimney. The elevation of the center line of these tubes is 45 ft.

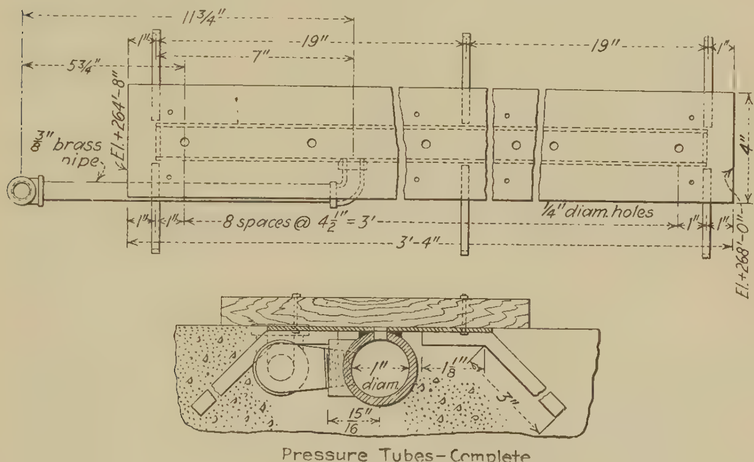


FIG. 6.—DETAILS OF PRESSURE TUBES.

below the elevation of the center line of the 12-ft. wind velocity tubes and 190 ft. above the base of the chimney. In order to register pressure accurately the tubes are attached to brass plates 4 in. wide and 3 ft. 4 in. high, the faces of which are flush with the face of the chimney. Openings in the tubes and in the brass plate are ¼ in. in diameter, spaced 4 in. on centers. The pressure in each tube is transmitted to the base of the chimney through a separate ⅜-in. pipe.

The 18 tubes were set in the chimney while it was under construction but it is not the intention to take readings until the 12-ft. wind velocity tubes are in place. Fig. 6 shows the detail of the tubes in the face of the chimney, the method of attaching them to the brass plates, and the arrangement for leading the ⅜-in. pipe through the concrete to the surface of the chimney and down to the base of the stack. Fig. 7 shows location of all thermo-couples, outriggers, tubes and strain gage points.

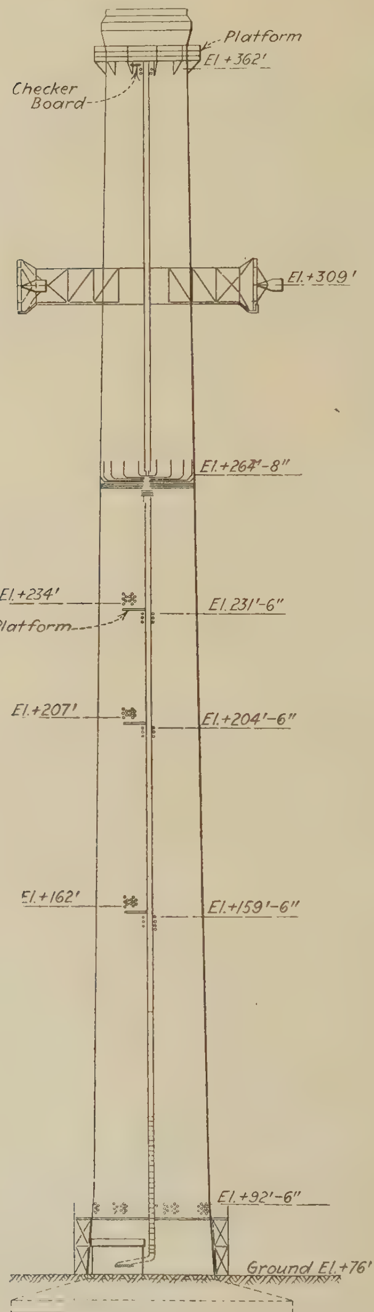


FIG. 7.—LOCATION OF THERMO-COUPLES, OUTRIGGERS, TUBES AND STRAIN-GAGE POINTS.

The four ⅜-in. pipe from the 12-ft. integrating impact pressure tubes registering wind velocity and the eighteen ⅜-in. pipe leading from the 3 ft. 2-in. tubes registering pressure on the chimney will be brought together alongside of each other at the base of the chimney onto a single tilting platform approximately 3 ft. wide and 4 ft. long. Glass tubes filled with colored liquid will form the manometers to register pressure in the tubes. The record will be made photographically.

A shack has been built at the base of the chimney to house the switchboard and potentiometer to read temperatures and also to house the manometers.

Chimney Deflection.

The nature and extent of the deflection of the chimney due to wind pressure will be determined by means of a telescope and mirror on an isolated foundation on the ground 44 ft. from the base of the chimney and a steel horizontal plate 16 in. square set firmly out from the chimney near the top of the stack. The underside of this steel plate will be painted like a checkerboard and will be illuminated so that the mirror at the ground level will reflect the checkerboard. The telescope will be rigid and any movement of the chimney and checkerboard will be read in the mirror. This method of reading chimney deflections has been used successfully on chimneys in Germany. Fig. 8 shows the arrangement for reading deflections.

Work to Date.

During the construction of the chimney 6 x 12-in. cylinders of the concrete from each section of the chimney were made for testing. The average strength at 7 days was 1,453 lb. per sq. in., at 28 days, 2,739 lb., and at 3 months 4,493 lb. The maximum and minimum variations at 7 days were approximately 33 per cent from the average, at 28 days 33 per cent, and at 3 months 25 per cent. The specifications called for a maximum slump of not more than 4 in. After the workmen became accustomed to this requirement good results were obtained. The mix used at first was 1: 2: 3 but was later modified to 1 cement, 2½ sand 2⅔ trap rock graded from ¼ in. to 1½ in. The strengths of the two mixes were practically the same but the workability of the 1: 2½: 2⅔ mix was better than the 1: 2: 3 mix.

Strain gage readings have been taken on the concrete specimens to determine shrinkage. These strain gage readings show an average shrinkage so far of 0.02 per cent. Wood plugs were left in the concrete shell of the chimney as the work progressed wherever it was necessary to later reach the reinforcing for strain gage points.

In order to provide for ease in obtaining strain gage readings, rigid platforms have been placed just below each set of strain gage points. The chimney ladder has also been caged for the entire height of the chimney. Close to the top a steel platform has been built all around the chimney for

future experiments. It will be a simple matter to reach any point on the outside of the chimney by attaching ropes to this platform. Fig. 9 is photograph of chimney at present.

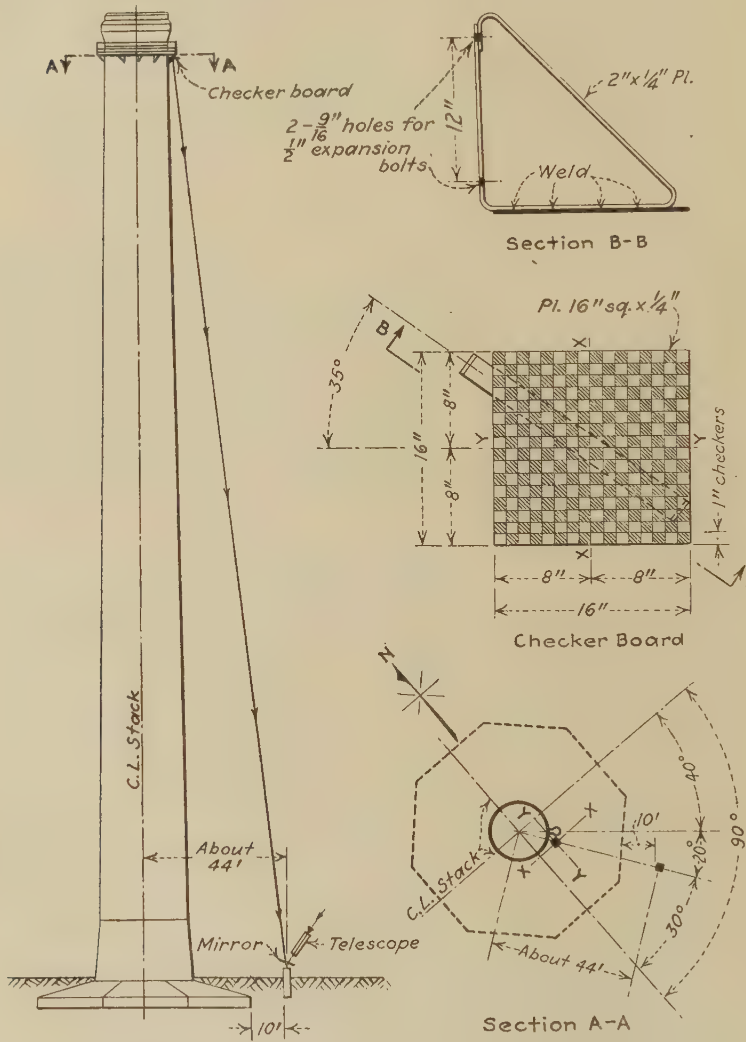


FIG. 8.—CHECKER BOARD ARRANGEMENT FOR READING DEFLECTIONS.

On Jan. 22, 1926, gas from one of the cement plant kilns at a temperature of 240 deg. and entering at the rate of 60,000 cu. ft. per minute was turned into the chimney but on account of the fact that the lower

breeching was still open, allowing free entrance of cold air, the temperature of the gas in the center of the chimney was only 60 deg. F. The gas is drawn from the rotary kilns and driers, and passes through a Cottrell dust-collecting system before entering the chimney.

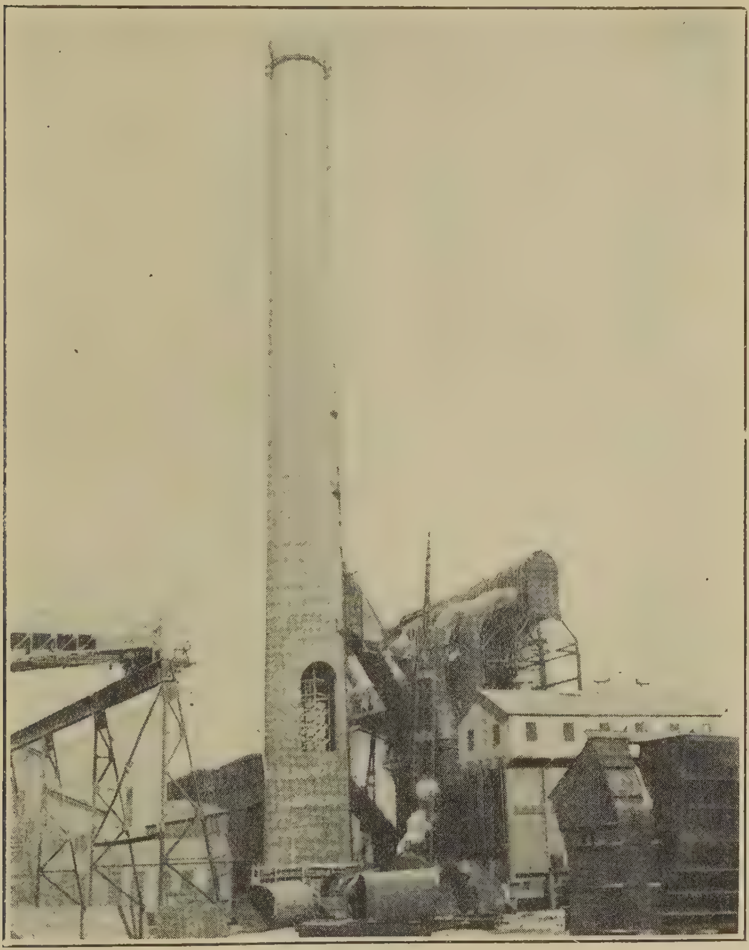
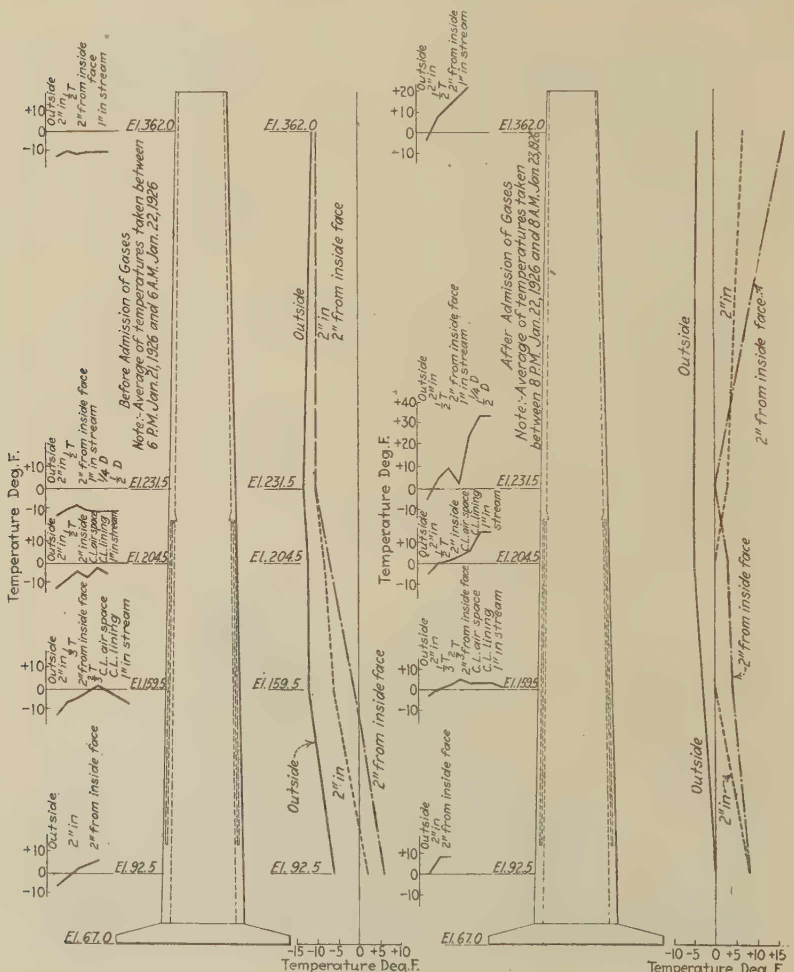


FIG. 9

Temperature readings were taken every hour for 28 hours before and for 72 hours after the gas was introduced into the chimney. Outdoor temperatures during this period ranged from a minimum of 14 deg. below zero to a minimum of 22 deg. above zero. The temperature in the con-

crete approximated the outdoor temperature within a comparatively few hours. It was interesting to note that the effect of the heat of the sun was quickly registered by the thermo-couples. The sun's heat raised the temperature of the concrete considerably above the air temperature. The



FIGS. 10 AND 11

temperature of the concrete even with no heat on the inside of the chimney was higher during the middle of the day with the sun shining than the temperature of the air just adjacent to the chimney. At the Duluth chimney it will be possible to cover a wide range of temperatures because cold

air can be introduced at the base of the chimney. This will be very helpful in studying temperature gradient when the gas is at different temperatures.

In view of the fact that the highest temperature of the gas on the inside of the chimney until now has not been more than 60 deg., no conclusions as to the maximum temperature gradient through the concrete shell can be given at the present time.

Readings will be taken when the breechings now open are closed and a full amount of gas is entering the chimney. At that time it is expected that the temperature of the gas on the inside of the chimney will be as high as 400 deg. Enough readings have, however, been taken to show that the temperature may vary considerably through the chimney shell. The location of the cold and warm air entrances also have considerable effect on the temperatures at a certain section. The temperature of the gas stream 1 in. from the concrete may be far below the temperature in the middle of the gas stream. Fig. 10 shows the average temperature at various elevations as recorded by the thermo-couples for a 12-hour period before the gas was introduced into the chimney. Fig. 11 shows the average temperatures for a 12-hour period after the gas was introduced.

The tests on this chimney are already attracting considerable attention because of their comprehensiveness and the unusual method of determining wind velocities and wind pressures.

Readings will be taken during the next twelve months and it is expected that the report to be presented at the 1927 convention of the American Concrete Institute will give conclusions as to the temperature gradient through the chimney wall, as well as data on wind pressures that will be fundamental in the design of chimneys.

The sub-committee on tests and research of the chimney committee, which is acting in an advisory capacity on these tests, is composed of E. A. Dockstader, of Stone & Webster; Prof. E. R. Maurer, of the University of Wisconsin; A. C. Irwin, of the Portland Cement Association; Prof. M. B. Lagaard, of the University of Minnesota; J. W. Lowell, of Benedict Stone, Inc.; and Benjamin Wilk, of the Universal Portland Cement Co.

EXTENSIBILITY OF CONCRETE*

By W. K. HATT.**

This paper records measurements of the shrinkage and expansion of concrete under various conditions of exposure, and presents a study of the ability of concrete to undergo extension without the appearance of surface fissures, both in the case of plain concrete and concrete reinforced with fabric reinforcing from 0.26 to 0.33 per cent. The concrete is 1: 2: 3 gravel aggregate and generally cured under unfavorable conditions.

Crack surveys of the surfaces of the beams classify surface fissures into (a) miscroscopic fissures about 0.0004 in. in width and (b) those seen with the unaided eye about 0.0015 in. in width.

When it prevents the natural shrinkage of concrete, reinforcing promotes tensile stress on the surface, and thereby tends to produce fissures. This effect is evident when reinforcing is in a structural amount from ¾ to 1 per cent. At low limits of surface reinforcing, for example 0.3 per cent, a mesh or fabric reinforcing aids to preserve the integrity of concrete surfaces. The reinforcement apparently does not postpone the appearance of microscopic fissures, but does postpone the appearance of eye-visible fissures.

The extensibility of 1: 2: 3 concrete before microscopic fissures appear is 0.010 per cent and at fissures visible to the unaided eye 0.018 per cent. Defective curing will reduce this latter value to 0.015 per cent. Concrete kept moist will extend farther before cracking than concrete kept dry. Mortar 1: 2 extends about 0.02 per cent.

The usefulness of mesh reinforcing is most evident under fatigue tests.

*The experiments recorded below were made in the laboratory for testing materials of Purdue University partly under a co-operative agreement with the U. S. Bureau of Public Roads.

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The extensibility of tests reported in the paper were made mainly on concrete cured under unfavorable conditions. An additional series of tests is now under way using several aggregates and favorable conditions of curing.

Without restraint concrete may shrink upon drying 0.05 per cent and swell when wet 0.01 per cent. Reinforcement may reduce these gross changes of length one-half. A change of temperature of 100 deg. F. will produce a change of length of 0.050 per cent. Restraint will modify these values, as for instance the friction of the subgrade under a concrete pavement or the restraint of a wet core upon the drying surface of a concrete beam. The ability of ordinary concrete to withstand surface deformations is below the measure of 0.02 per cent. Fissures are, therefore, likely to result from restraint, and the problem is to lessen surface movements by proper proportioning and curing, or by other means.

THE CLASSIFICATION OF SURFACE FISSURES.

While measurements of the strength of concrete are plentiful, data of toughness, that is its ability to withstand deformations without the appearance of surface fissures, are few. Since the permanence of concrete is largely dependent upon the preservation of an integral surface, the study of extensibility and the means of increasing extensibility is of importance

Surface fissures in concrete may be produced by load or by the action of temperature or moisture changes. They may range in magnitude: (a) from those in the order of 0.0004 in. width seen only with a microscope appearing as "water veins" or "water marks," as Feret termed them, (Étude Experimental du Ciment Armé, p. 51), when a skin-dried surface breaks and capillary moisture comes through the fissures; to (b) larger fissures in the order of 0.0015 in. width, seen by the unaided eye; and (c) in the extreme to those large open cracks that occur when the elastic limit of reinforcing steel is exceeded. Fissures are those crazes that mar the appearance of architectural concrete or other concrete products. Such crazes are not always evident to the unaided eye, but may be developed by a coating of light oil.

Climatic changes express themselves most markedly when the surface of the concrete is of a richer composition than the interior, or when the surface is contracted by careless drying against a moist core. Indeed the falling off in strength of cement briquettes that have been taken from the water and then exposed to the drying air of a laboratory before testing has been explained by the presence of surface fissures arising from the sudden shrinkage of the surface of the briquettes.

Crazes appeared in carelessly cured concrete exposed to the weather during summer appeared after two weeks of exposure, first on the top surface of 6 x 8-in. beams, three months later on the sides, and finally on the bottoms.

For example, the following observations were made on two series of gravel concrete beams of 1: 2: 3 mix, (a) cured in a shed and (b) exposed to the weather. Each was imperfectly cured concrete having been cured under damp burlap during only two days. Beams included both plain and reinforced specimens (0.25 per cent fabric reinforcement).

NOTES ON APPEARANCE OF SURFACE CHECKS UPON EXPOSURE.

Six-inch by eight-inch by five-foot, 1: 2: 3 gravel concrete beams were exposed in two series:

Series 1 exposed to the air of a shed after two days under wet burlap.

Series 2 exposed to the summer weather.

7 Days—None of the beams stored in the shed or outside showed any indication of check or craze cracks on the exposed finished surface even under the application of a thin film of oil.

14 Days.—Beams stored outside show slight check or craze cracks.

Beams stored inside show no check or craze cracks even under the application of oil.

21 Days.—Beams stored outside show considerable check or craze cracks.

Beams stored in shed show very slight check or craze cracks even upon the application of a thin film of oil. Failures under test do not seem to follow the check or craze cracks to any marked degree.

28 Days.—Beams stored outside show pronounced check or craze cracks.

Beams stored in shed show some check or craze cracks which become more pronounced when oil is used. Under subsequent loading test failures followed when possible the check cracks, especially in the case of the beams cured outside.

60 Days.—Beams stored outside show very pronounced check or craze marks, even the sides of the beams which have an unfinished surface are badly checked. Beams stored inside show slight indications of checking or crazing on finished surface which do not become plainly evident even when a coating of oil is used. Under subsequent test the failures seem to follow these check cracks very closely, especially in the case of the beams cured outside.

In the specimens used for fatigue tests at 28 and 60 days, the first microscopic fissures to open under load usually follow these check marks and a majority of the ultimate failures occur thereat.

It appears, therefore, that the surface checks or crazes develop more slowly in concrete exposed to the air of a shed than in concrete exposed to

the weather. These surface checks in the shed exposed concrete were at no time visible to the naked eye except upon the application of oil to the surface. As appears in Fig. 13, the shed exposed concrete showed greater

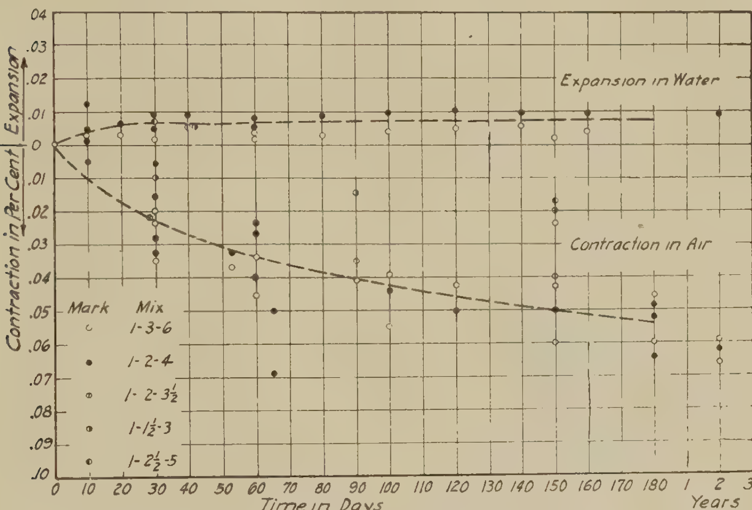

FIG. 1.—SUMMARY OF RECORDED TESTS OF CONTRACTION AND EXPANSION OF CONCRETE.

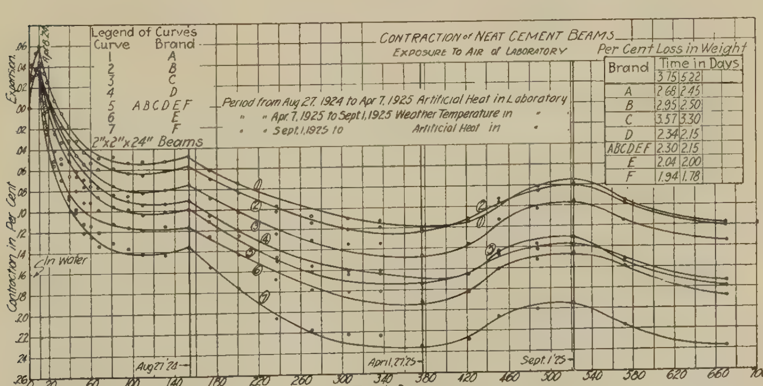

FIG. 2.—EXPANSION AND CONTRACTION OF SEVERAL BRANDS OF NEAT CEMENT. (Fig. 4 represents a six months later market sampling of cements A, D, F, G.)

extensibility at early ages and also at 135 days, but at the age of 63 days the measurements show a reversal of order. Four beams are included in each average. It may be noted that neither one of these concretes had a normal curing.

Six months.—No change in appearance of surfaces.

No distinct difference was noted in the crazing of the plain and reinforced specimens.

Further examination was made of the characteristics of the craze marks seen on concrete surfaces to determine if they represented fissures into the concrete.

The method was by the spreading of a one per cent solution of methyl violet dye in alcohol on the surface of the beams before they were broken in cross-bending. Subsequent examination of the fractured surface disclosed the penetration of the dye. If the crazes were real fissures, then the dye would penetrate into the body of the concrete.

A study was made by this method also of the development of surface fissures due to various degrees of extension of the surface under cross-bending.

The beams had been exposed to the weather for six months after three days curing under wet burlap.

A. The three intensities of surface crazing as determined by preliminary inspection of the beams was as follows:

1. Badly crazed.
2. Moderately crazed.
3. Free from crazing.

Two methods of troweling were represented:

1. Troweling subsequent to the initial set of the concrete.
2. Troweling of the wet surface.

The observations of the penetration of the dye indicated that the so-called crazes have no appreciable depth. They extend 1/64 to 1/32 of an inch only into the outermost cement skin, and did not enter the mortar or extend around the fine or coarse aggregate. They do not appear to be fissures.

The appearance of crazing is more pronounced on the surfaces troweled when wet. Only minor crazing existed when the surface was troweled subsequent to the initial set of the concrete.

B. Relation of extension at the development of fissures under cross-bending tests as shown by dye observations to the development of fissures visible to the eye:

This relation was determined as follows:

Previous to test, dye was applied to one-half of the tension face of the beams throughout the length of the stressed section for the determination of the depth of crazes as discussed above. Then during loading, when the extensometers by sudden progression indicated the development of a fissure, dye was applied to about 1½ in. of the remaining width of the tension face, and in similar widths for each added increment of load until eye-visible cracks appeared. Observations of the penetration of the dye on the broken sections after test determined the load at which fissures first developed. The relation is shown on the following table:

Beam	Fissure by Dye Indication		Eye-Visible Fissure	
	Load	Unit Extension	Load	Unit Extension
Single Reinf.	3400	0.000207	3400	0.000207
" "	3000	0.000137	3100	0.000205
Double "	3300	0.000174	3400	0.000200
" "	3050	0.000164	3200	0.000212
Plain	2800	0.00013	2800	0.00013
Plain	2800	0.00012	2800	0.00012

Weather Exposure—Age 6 months:

Three days initial curing under wet burlap.

6 in. x 8 in. x 5-ft. Beams, 1: 2: 3 Gravel Concrete 0.26 per cent R.

In the reinforced beam, 6 months of age, it was found that when fissures developed, the steel elongated resulting in the immediate formation of visible cracks and a rapid progression in extension.

In tests of beams at earlier ages, the load necessary to produce fissures was not sufficient to cause elongation of the steel to the extent of visible cracks in the concrete. It is believed that the use of dye in such tests would facilitate the detection of these minute fissures.

In the tests described below the main factors to be examined were *curing* conditions and the effect of *mesh reinforcement.*

THE EXPANSION AND CONTRACTION OF CONCRETE.

Prior to the research upon extensibility, a study was made of the effect of change of moisture conditions and temperature upon contraction and expansion and the warping and pulling of beams by change of condition of their surfaces. Values of contraction and expansion were measured and evidence of surface checking sought due to difference of condition of surface.* At that time no close survey was made of the beams to detect the first appearance of fine fissures, although these were evident in the surface of a road slab that had been warped back and forth by climatic effects.

A summary of the recorded measurements of expansion and contraction is shown in Fig. 1.

In this diagram are several variables:

Exposure due to changes of temperature, moisture, composition of cement, aggregate, wetness of mix, proportioning, size of specimen, forms, age of specimen, duration of test and time of beginning measurements.

The paper by the writer before the American Society of Civil Engineers (*Proceedings,* Am. Soc. C. E., May, 1925), endeavored to discuss the effect of these several variables on the expansion and contraction.

Composition of Cement and Exposure.

Figs. 2, 3, 4 and 5 show the results at Purdue University.

Figs. 6 and 7 show the important effect of wetness of mix and time elapsed as determined by Otto Graf (*Beton und Eisen,* 1921-22). The

*See *Proceedings,* Am. Soc. C. E., May, 1925.

importance of curing during the first 10 hours to reduce shrinkage is evident, and also the benefits of dry mixes.

Fig. 8 shows the shrinkage of 1: 3 mortar following different periods of curing in water as shown in *Der Eisenbetonbau*, Fifth Edition, Vol. I, Part 1, p. 136.

Fig. 9 shows the coefficient of expansion of concrete at various temperatures as determined at Purdue University. The coefficient increases with the temperature at which it is measured, and changes with the condition of the concrete in respect to moisture.

EXPANSION AND CONTRACTION OF STADIUM.

In the paper by the author before the American Concrete Institute in 1925 measurements were recorded of the expansion and contraction of concrete stadium at Purdue University. Since that time these measurements have been extended with the following results:

The top of the retaining wall between winter and summer expanded with a coefficient of .00000595 for one degree of Fahrenheit, and from summer to February, 1926, had contracted with a coefficient of .00000432. The difference between these coefficients is no doubt due to the moisture in the concrete during the winter weather. The average of the two coefficients is .00000513.

These same coefficients for the reinforced deck are .00000346 and .00000236, or an average of .00000291, which no doubt is a measure of the restraint of the columns and surrounding structure.

The coefficients for the deck on the fill are .00000547 and .00000267, or an average of .00000407, again an expression of constraint.

The Effect of Reinforcing.

The effect of reinforcing upon expansion and contraction is shown in Figs. 10 and 11. Fig. 10 shows the results of experiments by O. Graf and C. Bach (*Armierter Beton*, No. 9, 1909) and Fig. 11 recent tests at Purdue University. The effect of steel in reducing total shrinkage is evident. Of course tensile stresses in the concrete are set up, and these are greater as the amount of reinforcing is increased.

Reinforcing in general through its action in preventing the natural shrinkage of the concrete will promote surface fissures. Considere was apparently mistaken in his findings of 1898 that reinforced concrete would undergo greater extensions than plain concrete. Later tests by Bach and Kleinlogel in 1902 and at the University of Wisconsin in 1906 resulted in conclusions opposite to those of Considere. The values reported by Bach and Kleinlogel as found in *Der Eisenbetonbau* by Marsch are shown in Fig. 12. In the tests at Purdue University the reinforcing was in the amount of structural reinforcing. It remained to study the effect upon the surface of concrete of smaller amounts of fabric reinforcing, around 0.25 per cent such as are used in exposed surface.

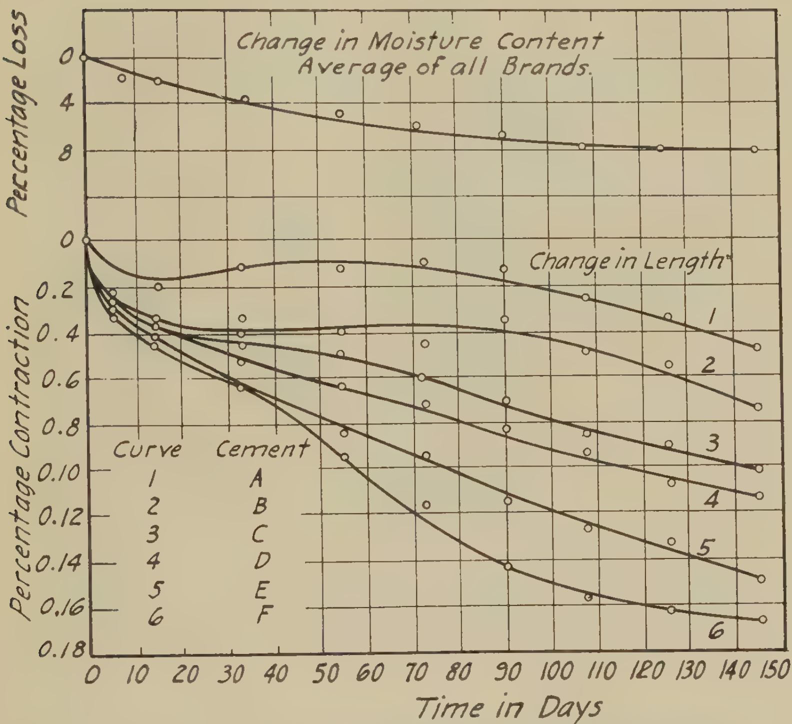

FIG. 3.—CONTRACTION OF NEAT CEMENTS OVEN DRYING.

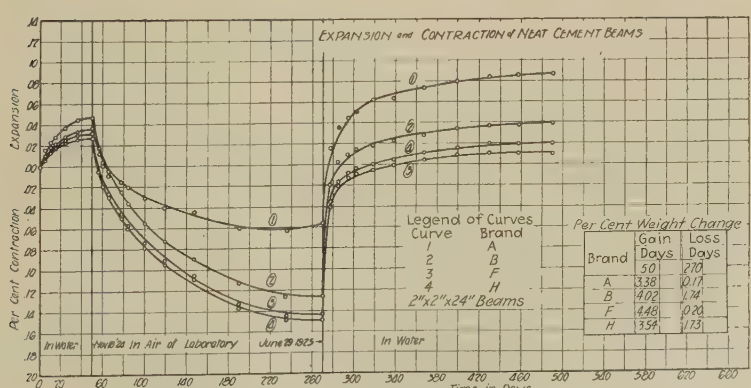

FIG. 4.—EXPANSION AND CONTRACTION OF NEAT CEMENTS.

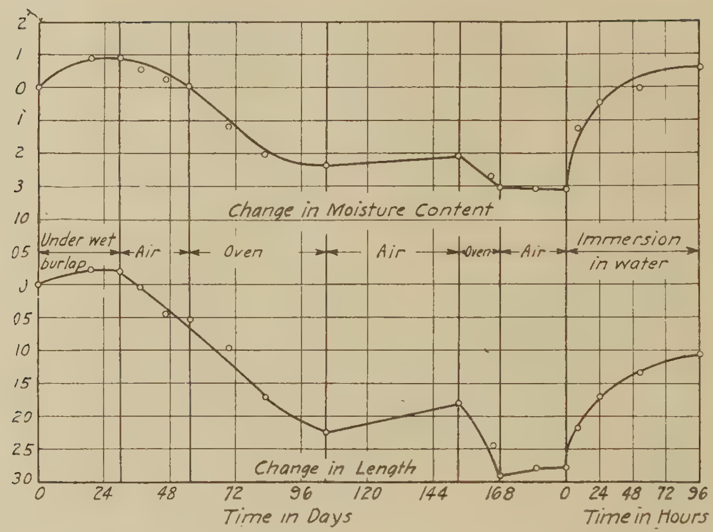

FIG. 5.—EFFECT OF EXPOSURE ON CHANGE OF LENGTH OF CONCRETE. MOISTURE CHANGES IN PER CENT. UNIT CONTRACTION 1 SCALE UNIT = 0.00001.

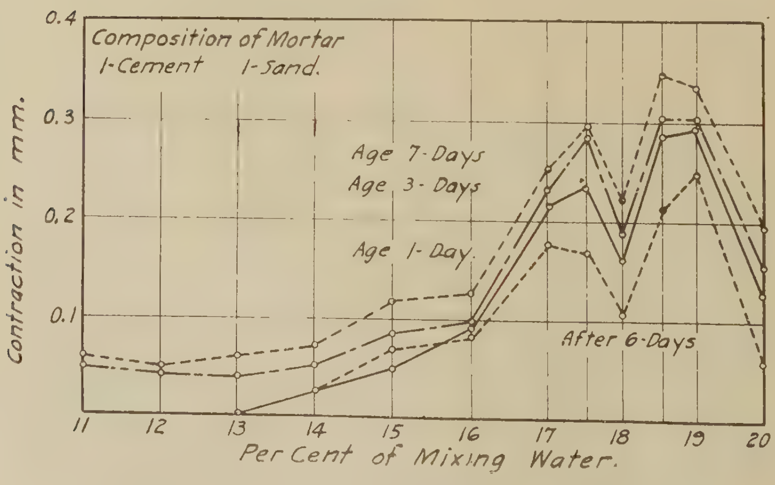

FIG. 6.—EFFECT OF AMOUNT OF MIXING WATER ON SHRINKAGE.

Tests by Graf.

TESTS OF EXTENSIBILITY OF PLAIN AND REINFORCED CONCRETE.

The present paper describes laboratory tests upon the extensibility of various classes of concrete, plain and reinforced with mesh reinforcement, under various conditions of exposure at different ages, both under the usual progressive loading and under repeated loading. Of special interest

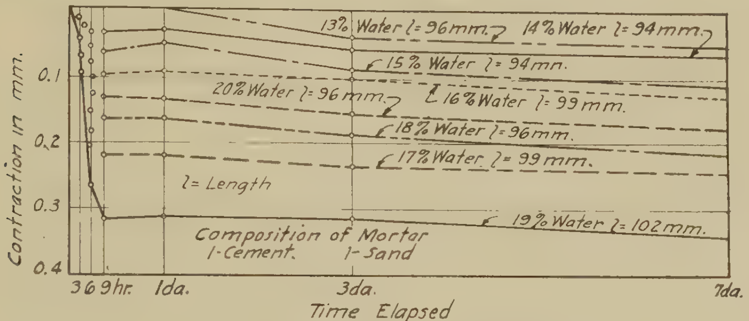

FIG. 7.—SHRINKAGE OF 1:1 MORTAR.

Tests by Graf.

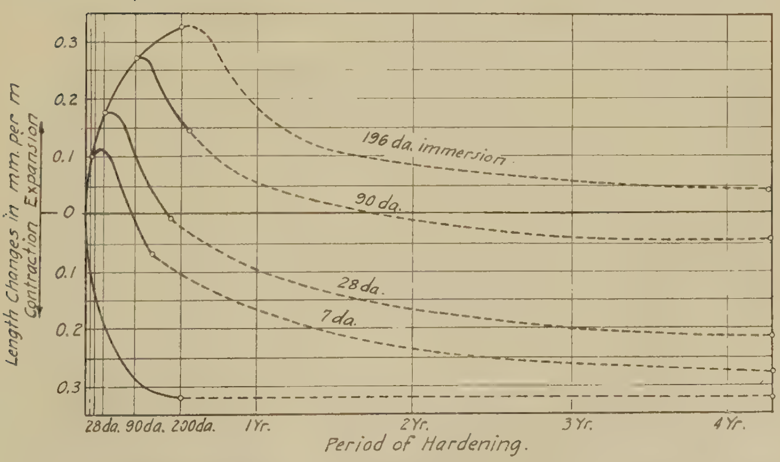

FIG. 8.—LENGTH CHANGES OF 1:3 CONCRETE AFTER INITIAL CURING IN WATER.

From *Der Eisenbetonbau*—Marsch.

is the study of the relative extensibility at the various ages of plain and mesh reinforced concrete. A paper by C. A. Hogentogler before the Highway Research Board, Washington, D. C., Dec. 4, 1925, reports the results of his survey of plain and reinforced-concrete road surfaces in actual service. The results confirm the laboratory tests.

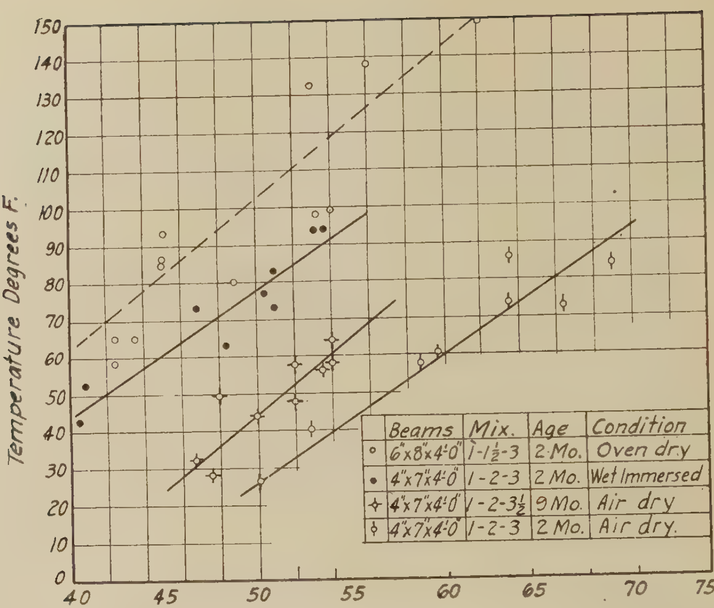

FIG. 9.—THERMAL COEFFICIENT OF EXPANSION OF CONCRETE AT VARIOUS TEMPERATURES. VALUES THUS: 50 = 0.0000050.

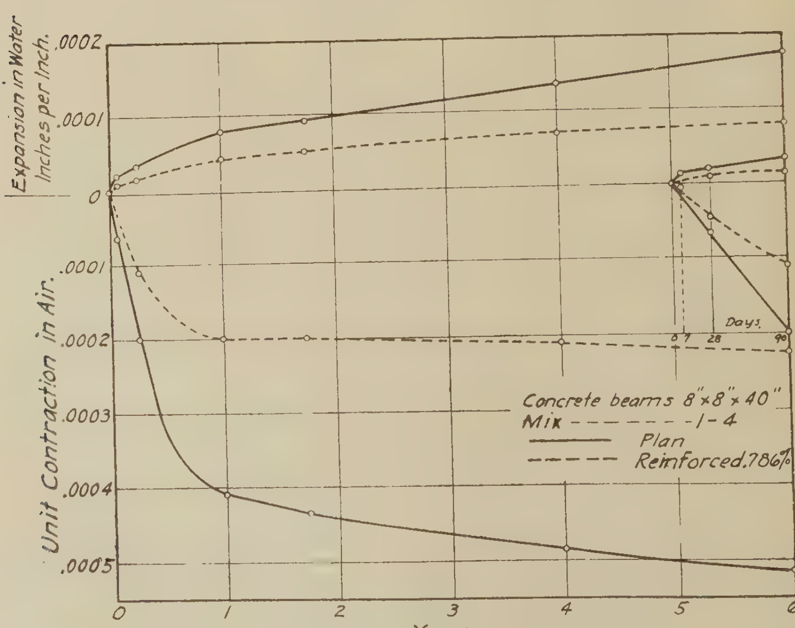

FIG. 10.—RELATIVE CHANGE OF LENGTH OF PLAIN AND REINFORCED CONCRETE.

Up to the present time the action of reinforcement is fairly well determined. However, severe conditions attending poor curing have been used in this latter investigation. Further studies now under way include

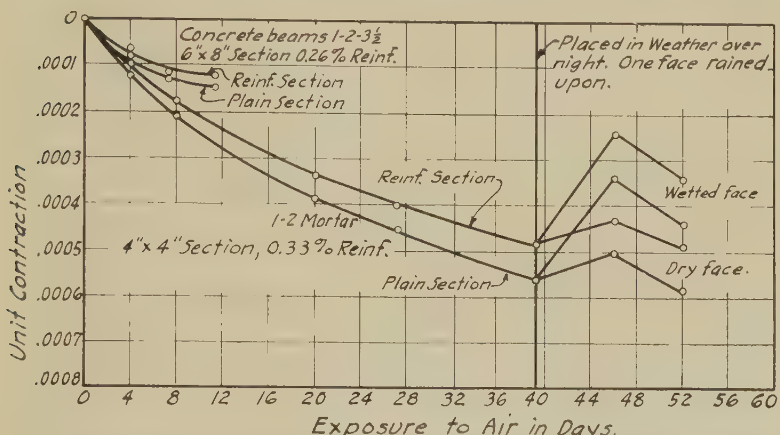

FIG. 11.—RELATIVE CHANGES OF LENGTH OF PLAIN AND REINFORCED CONCRETE.

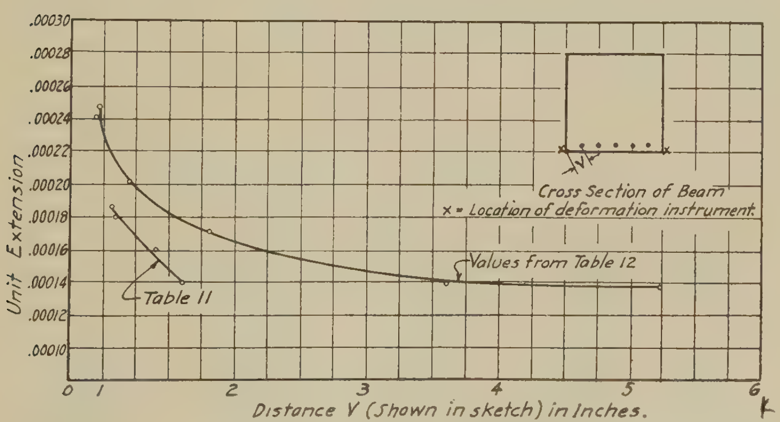

FIG. 12.—EFFECT OF DISTANCE OF REINFORCEMENT FROM THE EDGE OF THE BEAM ON THE UNIT EXTENSION. BEAMS APPROXIMATELY

Tests by Bach and Kleinlogel.

various kinds of concrete with more than one aggregate cured in several manners.

A preliminary series of tests was made in 1924 to gain experience in measurement. During the past season of 1925 the laboratory for testing

materials of Purdue University in co-operation with the U. S. Bureau of Public Roads has been investigating the extensibility of plain concrete and concrete containing mesh reinforcing as supplied by the National Steel Fabric Co. of Pittsburgh, Pa.

FLEXURE TESTS OF PLAIN CONCRETE BEAMS TO DETERMINE EXTENSIBILITY AT FIRST CRACK VISIBLE TO THE NAKED EYE.

The earlier tests involved a crack survey by the unaided eyes of trained observers. A light oil was often spread over the surface of the concrete to develop the appearance of checks and fissures. Later a 100-power microscope with a three-way motion was provided to search the surface of the 4 x 4-in. concrete specimens under repeated loads to determine the appearance of the fine microscopic fissures which generally occurred at an extension of approximately 55 per cent of the extension which accompanies the first crack visible to the naked eye. As has been said, such minute fissures even in plain concrete beams will open up and then spring back and close the fissure after the removal of the load. The first tests studied the transverse strength and extensibility of concrete beams cured first under wet burlap for a period of three days, then either (1) in air of shed; or (2) outside, exposed to summer weather. The age of the concrete at time of testing ranged from 7 to 63 days. The results of the extensibility and average strength of all ages is shown in Table I. The first crack is that visible to the naked eye. Each value represents two beams. Unavoidable variations in the positioning of the steel reinforcing no doubt will affect the results.

The deformation readings were taken with 20-in. Berry strain gages.

TABLE I.—STRENGTH AND EXTENSIBILITY OF PLAIN CONCRETE BEAMS OF 6-IN. DEPTH BY 8-IN. WIDTH, 54-IN. SPAN, 1:2:3 GRAVEL CONCRETE. Universal Cement: Slump 3½ in.

Age in Days	Curing	Modulus of Rupture	Unit Extension at First Crack	Deflection at First Crack
7	Inside	325	0.000155	0.0138
	Outside	334	0.000118	0.0088
14	Inside	420	0.000191	0.0156
	Outside	421	0.000136	0.0124
21	Inside	442	0.000164	0.0131
	Outside	470	0.000145	0.0125
28	Inside	410	0.000180	0.0144
	Outside	472	0.000127	0.0117
63	Inside	453	0.000135	0.0112
	Outside	590	0.000148	0.0137
135	Inside	503	0.000180	0.0127
	Outside	557	0.000166	0.0137

The extensibility of the beams cured outdoors increases somewhat with age and decreases with age during indoor storage although then the strength increased. The drier indoor concrete would naturally show less extensibility.

EXTENSION OF PLAIN CEMENT MORTAR TESTED UNDER FATIGUE AT PURDUE UNIVERSITY.

PURDUE UNIVERSITY. (Tests of 1924.)

The machine used and the methods of test are described in *Proceedings*, A. C. I., 1922, p. 167.

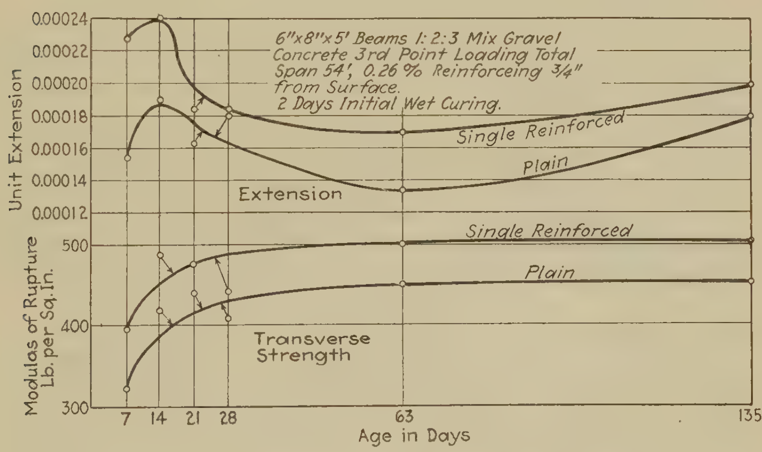

FIG. 13.—EFFECT OF AGE ON EXTENSION AND TRANSVERSE STRENGTH OF PLAIN AND FABRIC MESH REINFORCED CONCRETE STORED IN SHED.

In the fatigue tests upon 1:2 cement mortar a record of the extensions at failure has been obtained. Lehigh cement was used.

Fifty beams 4 x 4 in. in cross-section were broken under a single application of load and 50 were broken under repeated loads. The deformations were measured by means of Berry strain gages clamped on each side of a test piece. These specimens were all subjected to an initial curing of 14 days under wet burlap with subsequent exposure to the air of the laboratory. The age at test varied from 28 to 270 days, the majority of the specimens having been tested at an age over three months.

In these tests the unit extension at first visible crack of the mortar beams tested under the usual laboratory test using progressive loading to rupture averaged 0.000205; of the beams tested under fatigue loads the

unit extension averaged 0.000191. Beams of poorly cured 1: 2: 3 concrete, 4 x 4 in. in cross-section, tested in 1925, extended 0.000140 compared to the value of 0.000205 just quoted for 1: 2 mortar.

A test of relative extensibility of wet and dry mortar, 1: 2 mix, was made under progressive static loading using specimens of the shape designed for fatigue tests. The specimens were dry, 19 months old, or else were saturated by immersion during 8 days, before testing. The progressive static breaking load of the wet beams was 90 per cent of the dry specimens. The unit extension of the dry mortar at first crack visible to the naked eye was 0.00017 and of the wet mortar was 0.00020. Under fatigue these values of extension became 0.00015 and 0.00018 respectively.

TESTS OF PLAIN AND FABRIC REINFORCED BEAMS AT PURDUE UNIVERSITY (1925).

Tests conducted at Purdue University to determine the possible beneficial effect of a small amount of reinforcement on concrete and mortar beams have yielded measurements of extensibility, as listed in Table II, and shown in Figs. 13 and 14.

TABLE II.—EXTENSION OF PLAIN AND REINFORCED-CONCRETE BEAMS AT FIRST CRACK VISIBLE TO NAKED EYE.

Beams were 6 in. deep, 8 in. wide, 54 in. span, 1: 2: 3 concrete, singly reinforced with 0.26 per cent of fabric. Kept wet for 3 days after making, then air-stored inside the laboratory or else unprotected outdoors as indicated. Each value is the average of two beams. Slump = 3½ in. * indicates value for one beam only.

Age in Days	Mod. of Rupt. Plain	Mod. of Rupt. Reinf.	Unit Extension Plain	Unit Extension Reinf.	Deflection, Inches. Plain	Deflection, Inches. Reinf.
			Cured Outside Unprotected.			
7	334	408	0.000118	0.000178	0.0088	0.0151
14	421	446	0.000136	0.000185	0.0124	0.0142
21	470	471	0.000145	0.000193*	0.0125	0.0146*
28	472	499	0.000137	0.000189	0.0117	0.0159
63	590	549	0.000148	0.000188	0.0137	0.0154
135	557	555	0.000166	0.000153	0.0137	0.0131
			Cured Inside.			
7	325	396	0.000155	0.000227	0.0138	0.0181
14	420	487	0.000191	0.000240	0.0156	0.0175
21	442	476	0.000164	0.000185	0.0131	0.0158
28	410	420	0.000182	0.000185	0.0144	0.0127
63	453	501	0.000135	0.000171	0.0112	0.0127
135	503	467	0.000180	0.00020	0.0127	0.0128

A very clear difference in the behavior of plain and reinforced specimens appears.

It is apparent from Table II that the fabric reinforcement had increased the extensibility of both the beams cured outdoors and those cured indoors up to 63 days. The action is pronounced at early ages.

COMPARISON OF BEAMS WITH SINGLE AND DOUBLE REINFORCEMENT.

A comparison was made of the relative behavior of single and double reinforced beams in 1 : 2 : 3 concrete, stored outside exposed to the weather. The results are shown in Table III and Fig. 14.

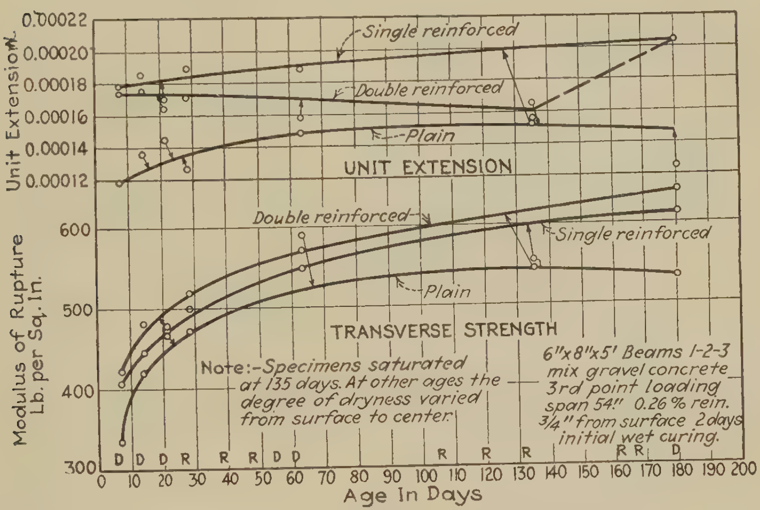

FIG. 14.—EFFECT OF AGE ON EXTENSION AND TRANSVERSE STRENGTH OF PLAIN AND FABRIC MESH REINFORCED CONCRETE EXPOSED TO WEATHER.

TABLE III.—STRENGTH AND EXTENSIBILITY OF SINGLE AND DOUBLE REINFORCED-CONCRETE BEAMS.

Cured outside. 1 : 2 : 3 Gravel Concrete, Slump 3½ in.

Age in Days	Reinforcement	Modulus of Rupture	Unit Extension at First Crack	Deflection at First Crack
7	Single	408	0.000178	0.0151
	Double	421	0.000174	0.0141
14	Single	446	0.000185	0.0142
	Double	480	0.000175	0.0146
21	Single	396	0.000170	0.0135
	Double	478	0.000164	0.0132
28	Single	499	0.000189	0.0159
	Double	520	0.000171	0.0139
63	Single	549	0.000188	0.0154
	Double	570	0.000158	0.0147
135	Single	555	0.000153	0.0131
	Double	549	0.000156	0.0125

These values show a greater strength but a less extensibility of the double reinforced beams as compared to the single reinforced at ages up to 63 days.

EFFECT OF REINFORCEMENT ON FATIGUE UNDER REPEATED LOADS.

A series of fatigue tests of 1 : 2 : 3 concrete reinforced with 0.33 per cent of mesh is typified in Table IV and in Fig. 15.

TABLE IV.—FATIGUE BEAMS IN FATIGUE TEST, SHOWING VALUES OF EXTENSION AND TRANSVERSE STRENGTH OF PLAIN AND FABRIC MESH REINFORCED-CONCRETE BEAMS.

4 in. x 4 in. x 30-in. Beams; 1:2:3 Gravel Concrete; 2 Days Initial Wet Curing, Then (1) Exposure to Weather (2) Storage in Shed; One Beam of each Exposure at each Age; Values Based on First Crack in Concrete Visible to Eye.

Age		Actual Values		Values in Per Cent		No. of Applications at Failure
		Extension	Mod. Rupt.	Extension	Mod. Rupt.	
7	Reinf.	0.000165	503	122.2	133	90,600
	Plain	135	378	100.0	100	40.850
14	Reinf.	155	492	107.0	123	136,600
	Plain	145	400	100.0	100	69,150
21	Reinf.	225	443	204.5	129	149,600
	Plain	110	343	100.0	100	29,650
28	Reinf.	167	463	104.3	143	232,400
	Plain	160	324	100.0	100	97,950
67	Reinf.	210	546	150.0	124	151,300
	Plain	140	441	100.0	100	45,350
140	Reinf.					
	Plain					
Aver.	Reinf.					
	Plain					

Table IV indicates that the eye-visible crack occurred at a greater extension in the reinforced than in plain concrete beams under fatiguing loads. The values are erratic since only one beam is represented in each value. Table V shows the measurements on a parallel set of beams to those tested under fatigue, when the loads were applied progressively as in an ordinary laboratory test. In this case also there was a large and more consistent increase in extensibility in the presence of reinforcing. Fig. 15 is a diagrammatic showing of the values in Table IV.

A very clear difference in the behavior of plain and reinforced specimens appears.

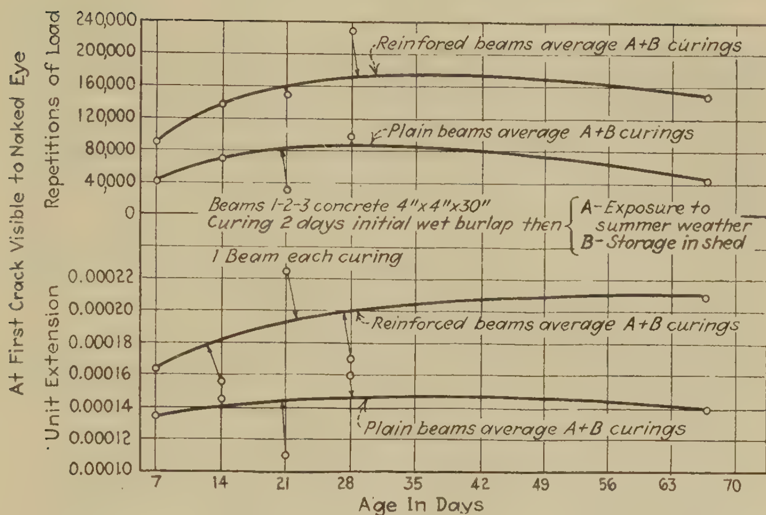

FIG. 15.—UNIT EXTENSION AND REPETITIONS OF LOAD ON PLAIN AND REINFORCED CONCRETE BEAMS UNDER FATIGUE TEST.

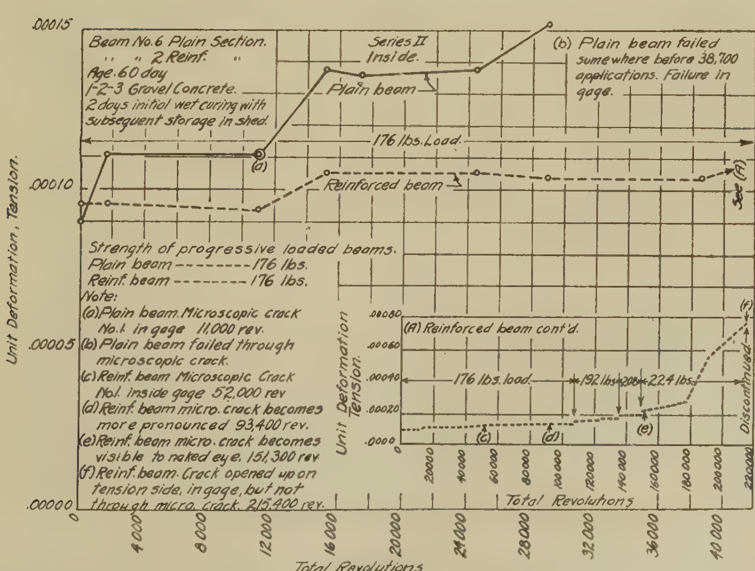

FIG. 16.—TYPICAL BEHAVIOR OF CONCRETE BEAM UNDER FATIGUE TEST.

The plain specimens break suddenly when the first eye-visible crack appears. When a crack forms in the stretched face of a reinforced specimen and the load is then removed, the reinforcing steel pulls the edges of the crack tightly together again, so that the crack is no longer open. The steel continues to draw the edges together until the elastic limit of the steel has been exceeded. Evidently steel of a high elastic limit is of advantage for this purpose. Minute capillary fissures will also close up upon removal of load in the case of plain concrete.

TABLE V.—FATIGUE BEAMS IN PROGRESSIVE LOADING, SHOWING VALUES OF EXTENSION AND TRANSVERSE STRENGTH OF PLAIN AND FABRIC MESH REINFORCED-CONCRETE BEAMS.

4 in. x 4 in. x 30-in. Beams; 1:2:3 Gravel Concrete; 2 Days Initial Wet Curing, Then (1) Exposure to Weather and (2) Storage in Shed; One Beam of each Exposure at each Age; Values Based on First Crack in Concrete Visible to Eye.

Age		Actual Values		Values in Per Cent	
		Extension	Mod. Rupt.	Extension	Mod. Rupt,
7	Reinf.	0.000215	590	165.3	146
	Plain	130	404	100.0	100
14	Reinf.	200	598	138.0	115
	Plain	145	518	100.0	100
21	Reinf.	160	487	97.0	105
	Plain	165	464	100.0	100
28	Reinf.	190	514	118.8	103
	Plain	160	499	100.0	100
67	Reinf.	205	783	110.8	103
	Plain	185	756	100.0	100
140	Reinf.	230	646	135.2	112
	Plain	170	577	100.0	100
Aver.	Reinf.	200	619.7	127.5	114.2
	Plain	159	536.3	100.0	100

The microscopic fissures in the face of the beam and their relation to cracks visible to the naked eye were determined in the fatigue tests and shown in Table VI. It appears that these microscopic fissures or water veins, as they were termed by Feret, occur at an extension of approximately 53 per cent in reinforced-concrete beams, and 67 per cent in plain concrete beams, of the extension at which eye-visible cracks appear. The number of repetitions of loads is approximately 40 per cent at the time of these microscopic fissures compared to the number of repetitions at the first visible crack.

The typical behavior of a beam under repeated stress is shown in Fig 16. The plain concrete beam showed microscopic fissures at 11,000

repetitions and a unit extension of 0.00011, and then failed at somewhere in advance of 28,000 repetitions and a unit extension of 0.00015. The reinforced beam showed microscopic fissures at 52,000 repetitions and the unit extension of 0.00012. Visible cracks appeared at 151,300 repetitions and at a unit extension of 0.00021. The test was stopped when a large crack approximately ⅛ in. in width opened at 220,000 revolutions.

Fig. 17 shows a similar chart of a typical test of a 1: 2 mortar beam, 16 days old, under fatigue.

TABLE VI.—RELATION OF MICROSCOPIC TO EYE-VISIBLE FISSURE

1:2:3 Concrete; Gravel Aggregate; Slump 3½ in.; 2 Days Initial Curing Under Wet Burlap; Fatigue Tests 4 x 4-in. Section; 0.33 per cent Reinforcement.

Age	Type of Beam	Storage	Microscopic Fissure		Eye-Visible Fissure	
			Unit Exten.	Applic. Load	Unit Exten.	Applic. Load
21	Reinf.	Laboratory	0.00012	55,800	0.00025	149,000
21	Reinf.	Weather	0.00011	65,300	0.00020	149,000
28	Reinf.	Laboratory	0.00010	186,000	0.00019	214,000
28	Reinf.	Weather	0.00009	75,000	0.00016	200,000
28	Reinf.	Laboratory	0.00010	109,000	0.00015	214,000
28	Reinf.	Weather	0.00008	88,000	0.00017	300,000
28	Plain	Laboratory	0.00011	14,600	0.00014	20,000
28	Plain	Weather	0.00010	47,000	0.00018	176,000
67	Reinf.	Laboratory	0.00012	68,000	0.00021	151,000
67	Reinf.	Weather	0.00010	42,000	0.00021	151,000
67	Plain	Laboratory	0.00011	11,000	0.00015	39,000
67	Plain	Weather	0.00009	39,000	0.00013	52,000
Average Values in Per Cent			0.00010 57.6	66,800 44.1	0.00018 100	151,350 100

SHRINKAGE MEASUREMENTS ON PLAIN AND REINFORCED BEAMS.

When steel reinforcing bars prevent the natural shrinkage of a concrete surface they bring about tensile strains in the concrete. These strains will increase as the steel bars increase in size. There appears to be a lower limit, however, where small bars in the form of fabric will serve to hold the concrete together and postpone the appearance of cracks visible to the naked eyes. Fig. 11 shows the degree to which 0.26 per cent of reinforcing caused restraint in the surface of these beams.

TESTS OF WELL-CURED CONCRETE.

In a preliminary series of tests of 1: 2: 3 gravel concrete in 1924 the relative deflection at first eye-visible crack of well-cured and also poorly-cured concrete beams was determined. One set was cured under wet burlap

until the time of test, and the other cured under wet burlap for only 2 days and then exposed to a continuous current of hot air at a temperature of 70 deg. F. The beams were 6 x 8 in. and 5 ft. long, some plain and some reinforced with steel mesh to 0.26 per cent. Table VII states the value of the deflection of the dry-cured plain beams to be 85 when the deflection of the wet-cured plain beams is 100.

Reinforcement appears to ameliorate the surface conditions caused by defective curing, especially in the case of the beams mixed to a 5-in. slump, when it has raised the extensibility of the dry-cured beams above that of the wet cured. [The strength of the air-cured beams is less in all cases.]

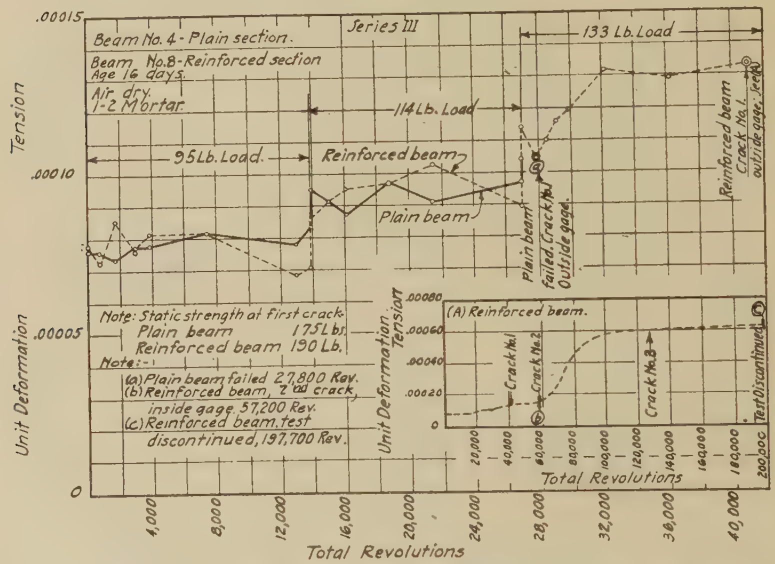

FIG. 17.—TYPICAL BEHAVIOR OF 1: 2 MORTAR BEAM 16 DAYS OLD UNDER FATIGUE TEST.

A preliminary series of tests on 1: 2 mortar cured in the manner just related for 1: 2: 3 concrete gave eight comparisons on the basis of specimens of the type used in fatigue tests. When tested under progressive loading the unit extension of the dry-cured beams was 77 per cent of the extension of the well-cured beams.

This same value of 77 per cent was obtained also for beams tested in fatigue. The cone slump of this concrete was 1¼ to 2½ in. Both plain and reinforced mortar enter into this average of 77 per cent.

The significance of minute fissures compared to those visible to the unaided eye in terms of service of concrete under load and exposure is obscure. Judging by tests made in Germany by E. Probst (*Armierter*

Beton, Vol. 12, p. 105, May, 1919) to determine the durability of reinforcement in concrete beams under load in an atmosphere designed to accelerate rust producing conditions, the very fine checks which accompany an elongation of 0.010 per cent have no significance in terms of rusting of steel. From the observation of the surface of concrete roads where these checks are undoubtedly present, it would not appear that they have much significance in respect to the wear of the surface of the roads.

TABLE VII.—RELATIVE DEFLECTION AT FIRST EYE-VISIBLE CRACK OF DIFFERENTLY CURED PLAIN AND REINFORCED-CONCRETE BEAMS OF 1 : 2 : 3 GRAVEL CONCRETE

Beams were cured (a) under wet burlap, and (b) in a continuous current of hot air. The deflection of the well-cured plain and reinforced beams is each expressed as 100.

Age in Days	Slump in Inches		Relative Value of Air-Cured Beams When Wet Cured = 100 Strength	Deflection
14	⅞	Plain	84.2	93.5
		Reinf.	90.8	98.0
14	5	Plain	73.2	87.0
		Reinf.	83.5	125.0
28	⅞	Plain	78.0	69.0
		Reinf.	83.7	102.0
28	5	Plain	94.5	86.0
		Reinf.	98.5	145.0

The conclusion from the data of the experiments is that mesh reinforcement will postpone the appearance of the eye-visible fissures and lessen the defects arising from poor manufacture and curing of concrete. It follows that reinforcing in small amounts should be of substantial advantage in maintaining the integrity of concrete surfaces. The investigation covered concrete made with gravel aggregate and unfavorable curing conditions. Additional tests now under way employ several aggregates and normal curing.

What Are the Most Significant Tests for Concrete?

By A. T. Goldbeck.*

Anyone who can definitely answer the question propounded by the title of this paper is indeed wise in concrete lore. So wide are the ramifications of the various phases of materials, manufacture and uses of concrete that it is difficult if not presumptuous for me to attempt a full discussion of the significance of the various tests or to point out those which are most important. A full discussion, however, by those who have worked in the specialized departments of concrete should supplement and correct my own shortcomings as they will be revealed in the present paper.

The Aim of Concrete Tests.—Stated in a general way, and eliminating special research, all tests made in connection with concrete are aimed at the final production of concrete suitable for given purposes. The purposes vary with the type of structure and the service which that structure must render. Before discussing concrete tests, it, therefore, will be necessary for us to have clearly in mind just what kind of resistance concrete should have, the general nature of the service it must render, and the desirable properties of concrete which will best perform that service. Then we shall be able to consider the tests suitable for evaluating those properties.

All of us, with a little patience, could catalog the many uses of concrete and most of us would be surprised at the length of our list. If we were to analyze each kind of structure from a service standpoint we should find that here the concrete is called upon for one kind of service, there for an entirely different kind and many structures would have service requirements in common. For instance, the main service of a protected column in a building is to resist constant or practically constant compression of a predetermined amount; a sea wall is called upon to resist the abrasive action of ice and water-borne abrasives; the mechanical and chemical action of salts in solution; stresses due to wave action; alternate freezing and thawing, wetting and drying; temperature changes and static stresses. Culverts might have to withstand constant static loads, freezing, scour and surface abrasion and impact of traffic; pavements must resist direct tension, cross-bending, direct compression and shear, surface abrasion and the influence of moisture changes, freezing and thawing and impact of traffic—a very severe combination of stresses indeed. In other uses such as stucco the material might be almost independent of the influence of imposed loads but be called upon for high resistance to conditions of weather exposure, involving temperature and moisture changes and freezing. In still other structures resistance against alkali might be the most important consideration. Similarly, we might enumerate the kinds of service

*Bureau of Engineering, National Crushed Stone Association.

to be rendered by the concrete used in various structures. We would find in the end that concrete is called upon to have high resistance along certain well-defined lines as follows:

1. Compression
2. Tension
3. Cross-bending
4. Shear
5. Impact
6. Surface abrasion
7. Chemical action
8. Freezing
9. Stresses from alternate wetting and drying
10. Absorption
11. Permeability
12. Heat Resistance

Constantly applied or repeated

In some structures only one of the above resistances would be important; in others, still another would govern, while it is not impossible that some structures might require high resistance along all of the above lines. With the knowledge that concrete might be called upon to have resistant properties in any one or all of these directions, it becomes pertinent for us to inquire what sort of tests we should actually employ to insure concrete of the necessary quality for the particular service it must render.

At first thought one would undoubtedly conclude that the most sensible test or tests to use are those which most directly reveal the value of the particular qualities concrete should have for a given kind of service. Such tests might be termed primary tests. But, on the other hand, every concrete materials engineer knows that the direct or primary test for some of the desired properties of concrete are involved, time consuming, expensive and in some cases not well co-ordinated with service behavior. When the relation between service and test results is indefinite, naturally, the test results have no definite meaning and the value of the test is questionable. It must also be borne in mind that some tests are very much more easily performed than others and that there often is a direct relation between one quality and another. It thus becomes possible to test the value of the particular quality of concrete which is required in the structure by making tests on another or other qualities. Such indirect tests might be termed secondary tests. Immediately, therefore, we are led to a consideration of the relation which the various properties of concrete bear to one another.

The Compression Test.—The compression test is the most widely accepted of all tests as the criterion of the quality of concrete. No question can properly be raised as to the suitability of this test as applied to concrete which is to be subjected to compressive stress. But this test is also used to determine the quality of concrete which must be resistant to other kinds of stress. It seems desirable, therefore, to ask ourselves how far we are justified in accepting this quality as being indicative of the other

qualities concrete must possess under different forms of service. Let us attempt a comparison of the compression test with other tests for concrete.

The Tension Test.—Dr. W. K. Hatt, in his recent bulletin on "Researches in Concrete," quotes the following results between compression and tension from tests by O. Graf.

RELATIVE EFFECT OF AMOUNT OF MIXING WATER ON STRENGTH IN COMPRESSION AND TENSION

Per cent Water by Weight	Ultimate Strength Compression	Tension	Relation of Tension to Compression
10.0	2500	259	10.4 per cent
9.0	3000	255	8.5
7.8	3360	285	8.5
6.8	4100	300	7.3

It will be noted that although the decrease in compression with 3.2 per cent increased mixing water is 1,600 lb., or 39.1 per cent, the decrease in tension value is 41 lb., or only 13.6 per cent.

The above table shows the variation in compressive and tensile values as the mixing water varies. It is seen that the relation of tension to compression is not a constant. Since there is no constant relation of tension to compression, even when the aggregates are identical in the mixtures compared, it seems entirely unlikely that there can be any fixed relation between these values in concrete containing many variables. Tension results on concrete are very meagre and in view of the above it would seem quite advisable to make tension tests on concrete whenever resistance to direct tension is an important property. I feel that high resistance to tension is a very important property in concrete highways after the curing period has elapsed. We should attempt to get more information as to the tensile resistance of concrete, especially at early ages, and the many variables which influence this property should be investigated. These variables include the moisture condition at time of testing, characteristics of both fine and coarse aggregates, characteristics of cement and many others.

Cross-Bending.—Concrete for highways is the most important example of concrete which must have high resistance to cross-bending. Tests have been made by a number of investigators on the relative resistance of concrete to compression and bending. Those reported by Prof. Abrams in Lewis Institute Bulletin 11, 1922, are rather typical. The following table, made up from that bulletin, shows the lack of any constant relation between these properties.

While the compressive strength was lowered 65.0 per cent the flexural strength was decreased only 21.8 per cent. Tests made by Wm. B. Fuller and reported in the American Society of Civil Engineers *Transactions* for 1907 also show no fixed relation between compression and flexure. Still

other investigations bear out these results. We are, therefore, forced to the conclusion that where high cross-breaking strength of concrete is an essential quality, we cannot rely on the compression test to tell us whether or not we are obtaining concrete of the particular quality desired, and we had better resort to a cross-breaking test. It is true, that, in a general way, high compression goes hand in hand with high flexural resistance, but we often need more definite knowledge than that. For instance, in concrete road design we should know the value of the modulus of rupture in order that we might select a safe unit stress. Fortunately the cross-breaking test is even more simply performed than direct compression and is much more susceptible of use in the field than compression. The cantilever form of specimen now advocated in some localities is particularly convenient, for with it the same specimen may be broken in several places.

Per cent Mixing Water	Compressive Strength	Flexural Strength	Ratio: Flexural Strength / Crushing Strength
11.0	3760	575	0.153
11.3	3280	590	0.180
11.7	3100	570	0.184
12.1	2720	560	0.206
12.5	2580	550	0.212
13.8	1920	500	0.260
16.7	1300	450	0.347

Shear.—The direct shearing resistance of concrete is a quantity of minor importance, for it is almost impossible to develop the maximum shearing resistance of concrete without failure taking place in some other form of stress such as diagonal tension. A test for direct shear is not a particularly simple test to perform satisfactorily and such results as we have show it to be a high value not far from ¾ of the compressive strength. It seems certain that concrete having high compressive strength also has high shearing strength and as the exact shearing value of concrete is relatively unimportant a compression test will give a close enough indication of the resistance to shear for all ordinary conditions. Even for those rare cases where direct shearing resistance might be important, the compression test should serve with sufficient accuracy.

Impact.—Impact resistance of concrete is an involved subject just as is the abstract subject of impact. In discussing impact, the exact conditions under which the impact is delivered must be known. Thus, the size and shape of the test specimen, its support, the mass of the striking weight, its velocity at impact, the character of material in the striking weight, its shape, and perhaps many other factors enter to influence the results. If the resistance of concrete to impact is desired, the tests would have to be made under conditions as nearly approaching those of practice as possible, else the results will not be of any great value. As an illustration, the

form of impact test used by H. S. Mattimore[1] as a measure of wearing resistance of concrete would be of no value for determining the impact resistance of concrete slabs under motor truck traffic. Here the form of test used by the U. S. Bureau of Public Roads[2] would be more suitable. Again, whether the specimen is subjected to flexure or direct tension or compression under impact will influence the results. Further, the structure might be subjected to repeated impact of small amount as against a comparatively few impacts of large amount. In a general way, concrete having high compressive strength will have high resistance to most varieties of impact, and with our present knowledge, the compression test would seem to be the test best adapted as a general measure of impact resistance.

Surface Abrasion.—The principal surface abrasion tests thus far made on concrete include:

(1) The Talbot-Jones Rattler.
(2) The Mattimore Impact Wear Test.
(3) Miscellaneous Wear Tests.

Under miscellaneous tests might be included the use of a concrete ring specimen mounted in a brick rattler and subjected to the abrasion of a charge of shot.[3] The standard paving brick rattler has also been used with a specimen consisting of spheres of concrete.

A test for surface abrasion is important only on concrete which is to be subjected to abrasion. A concrete highway furnishes the best example of surface abrasion, as do also the concrete pavements in railroad stations.

Undoubtedly, the most comprehensive series of tests ever made to correlate the actual behavior of concrete under service conditions with the various tests of the quality of concrete are those made by the U. S. Bureau of Public Roads.[4] Twenty conclusions were drawn from these tests which need not be presented here. It is shown, however, that surface wear is affected by a great many factors. The important conclusions from our present standpoint are as follows:

1. "That the Talbot-Jones wear test is not, in general, an indication of the wear which takes place under traffic."
2. "That neither the crushing nor the transverse strength of concrete is a measure of its wear-resisting properties."

It seems quite evident that we cannot make any of the common laboratory tests on concrete to determine how it will actually wear under traffic. The conclusions from the Bureau of Roads Tests, however, furnish an excellent guide as to how doubtful materials will behave in an actual road surface under wear.

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[1] American Society for Testing Materials Proceedings, 1920, Part II, page 267, Concrete Aggregate for Highways, by H. S. Mattimore.
[2] Public Roads for April, 1924, Impact Tests on Concrete Pavement Slabs, by L. W. Teller.
[3] American Society for Testing Materials Proceedings, 1917, Part II, page 394, Tests of Concrete Aggregates, by J. P. Nash.
[4] American Society for Testing Materials Proceedings, 1924, Part II, page 864, Accelerated Wear Tests of Concrete, by F. H. Jackson and J. T. Pauls.

Mr. Mattimore's test was, unfortunately, not used on the Bureau of Public Road specimens and I am unable to compare it with traffic effects. In general, it seems that high quality concrete, sufficiently workable to produce a smooth finish is essential for uniform wear. Uniformity in aggregate is likewise desirable, and apparently the Deval abrasion test for stone is of some value, as it seems to be the case that stone having a per cent of wear greater than 7.0 wears faster than the mortar. There is need for the development of a suitable abrasion test for both gravel and slag.

Chemical Action.—Chemical action on concrete covers a wide range of materials, among which are alkali and sea waters, animal and vegetable oils, tanning liquors, sulphite liquor, vinegar, sugar solution, molasses and acids of various kinds. Considerable investigation has already been performed to discover the action of various chemicals on concrete, and the effect of a number of substances is summarized in Appendix 17 of the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, 1924. High quality concrete is not always effective in resisting chemicals but is more or less effective for others. Certainly where the destructive action of chemicals is to be resisted, concrete of the very highest quality is necessary and low absorption combined with high strength seem to be important characteristics. It should go without saying, however, that the action of doubtful chemicals should be investigated and reliance should not be placed alone on those tests such as compression and absorption which indicate concrete of high quality for chemical resistance.

Freezing.—Although much investigation work has been done on the effect of freezing on building stone, brick, and to some extent on drain tile, results on concrete are very meagre and the research now under way at the U. S. Bureau of Public Roads will furnish a welcome addition to our present small fund of information. Resistance to freezing seems to be dependent on a number of factors, including strength of concrete, absorption, amount of fine material, character of aggregates, richness of mixture, consistency, and no doubt others. One very important factor in the effect of freezing is the amount of water available for freezing. A specimen, saturated when placed in a freezing temperature will not necessarily be affected by freezing unless additional water is supplied to keep the specimen saturated during the freezing process.

It is felt that the technique of freezing and thawing tests, as such, or in their artificial form involving sodium sulphate crystallization within the pores of the concrete, is not satisfactorily developed. At the present time we had better resort to secondary tests and where high resistance to freezing is necessary the concrete should have, (a) high compressive strength, (b) low absorption. Moreover, tests on the aggregate should indicate (a) a low percentage of fine material passing the No. 100 sieve, (b) high resistance of the coarse aggregate to the sodium sulphate test or to a proper freezing and thawing test and low absorption.

Alternate Wetting and Drying.—It is now well established that con-

crete expands when wet and shrinks when dry. It has also been demonstrated that the richness of the mix and the character of the cement have much to do with the amount of movement which takes place. If the concrete is confined and is subjected to change in moisture, stresses are set up, now compressive, now tensile. For instance, in a highway, high shrinkage develops frictional resistance between the slab and the subgrade and tension is produced in the concrete. High moisture at the bottom of the slab and drying out at the surface produce bending stress.

Prof. Alfred H. White, of Michigan, tells us that concrete grows in length under conditions of exposure, and there seems to be no question that this effect combined with temperature effects has often resulted disadvantageously. In most cases a proper design of the concrete structure can be made to care for this effect of moisture, and it is not apparent that any particular test need be made on concrete to investigate this property. Where moisture-shrinkage and expansion are important, as they are in concrete highways, there seems to be good reason for believing that a shrinkage or expansion test on portland cement might be of very real value.

Absorption.—Resistance to absorption is an important property of concrete under many conditions. Certainly low absorption is a vital property of concrete exposed to the weather. High strength and low absorption are not necessarily complementary properties and therefore an absorption test on concrete might well be required for severe exposure conditions, especially where there is almost constant moisture which is likely to freeze. Observation of lean and highly absorbent concrete exposed to continuous moisture and freezing certainly has indicated its lack of permanence under these conditions, while richer mixtures were not similarly affected. Evidently the absorption test is also an important one in connection with certain concrete products such as concrete blocks, for the quality of proper "suction" is a significant one with such products.

Permeability.—One of the most complete series of permeability tests thus far published are those by Prof. M. O. Withey, of Wisconsin. From his tests it seems evident that, assuming proper curing, there is a rather decided relation between permeability and crushing strength, and properly cured broken stone or gravel concrete having a compressive strength of 2,500 lb. per sq. in. is water-tight. Insufficient curing, however, increases the permeability, but might not appreciably decrease the compressive strength. Apparently, also, a well-graded aggregate following Fuller's curve is of help in producing impermeable concrete. With sufficient curing, it might be assumed, therefore, that high compressive strength in concrete having well-graded aggregate, sufficiently insures its permeability.

Fire Resistance.—Fire resistance is dependent on a number of factors. Thus, the character of both the fine and the coarse aggregate is very important and so also is the richness of the mix. Certainly fire resistance of concrete is a special property dependent on things other than compressive strength or any of the other tests thus far mentioned. Unless information already exists on the fire resistance of any particular aggre-

gate which is being considered, it would be well to have special fire tests made on concrete containing that aggregate, for the fire resistance of concrete can be judged in no other way.

Resistance to Repeated Stress.—It is a fact that our information on the effect of repeated stress in bending on concrete is far from complete notwithstanding the very excellent work of Dr. Hatt at Purdue University, of H. F. Clemmer at the Illinois Department of Public Works and Buildings, and by Dean A. N. Johnson, of the University of Maryland, and the older tests in compression by J. L. Van Ornum at Washington University at St. Louis, and H. C. Berry at the University of Pennsylvania.

How is resistance to repeated stress affected by the characteristics of the aggregate and the moisture condition of the specimen? Will repeated compressive stresses follow the same laws as repeated tensile or cross-bending stresses? Have we any reason to believe that the resistance of concrete against repeated stresses applied hundreds of thousands of times can be measured by its resistance against a single load applied once? At present we cannot answer these questions. In general, it seems to be the case that high quality concrete as measured by the compression test, tension or cross-bending shows high resistance to fatigue, and that probably is as definite a statement as should be made. More study should be given to repeated stresses, particularly as their effects are influenced by the aggregates and moisture condition and too much reliance should not be placed in static load tests made on specimens in a given moisture condition for determining the resistance of concrete to fatigue, especially when tested under an entirely different moisture condition.

No doubt other tests for special properties of concrete such as thermal conductivity, bond, consistency or workability and likewise tests for the constituent materials would be well worth discussing. But enough has been said to indicate the importance of giving very careful consideration to the service to which concrete will be subjected, for the service should determine the particular properties to be built into the concrete. Special properties in many cases require special tests, while in some cases the desired properties may be foretold by means of secondary tests more simply made than primary tests.

In a general way high resistance to compression is an indication of high quality concrete, but it is often necessary to have more exact information on the particular qualities of concrete intended for a given purpose than are shown by the compression test. In those cases tests should be made for the specific qualities desired.

DISCUSSION.

Mr. Lindstrom. ROBERT SETH LINDSTROM.—That was a very interesting paper, and I wish to give you some of my experiences in the field. Some three years ago we had some sand on the job called torpedo sand. It seemed somewhat dirty and with the colormetric test applied, everything seemed to be all right as to cleanliness. It was sent to the Robert W. Hunt Laboratory, and was found to meet with the Fuller test as to the grading of same. After taking sand from six jobs in the city of Chicago sold by six different dealers and sand coming from six different gravel pits, the only variation was in passing the 14-mesh sieve with only 2 per cent off from the Fuller formula. So the sand we are getting in Chicago as torpedo sand is practically perfect as to grading, except that the Fuller formula is that no percentage shall pass a 100-mesh sieve, while these sands taken from the actual job show that 3 per cent does pass the 100-mesh sieve, which practically substantiates some of the arguments here that at least 5 per cent of fine sand for getting good concrete should pass a 100-mesh sieve.

That fineness may be taken care of partially in portland cement, when you take portland cement of standard fineness of 78 passing a 200-mesh sieve, you get a 22 per cent residue on the sieve. That residue you call inert matter, or sand forming part of the percentage passing a 100-mesh sieve.

The question was brought out as to the abrasion and compression test. That is a very vital point, if we take into consideration the paper read by the man from Washington on earthquakes, which shows that we will have compression to contend with in foundations and walls against earthquakes, where quakes are of consideration. Personally, I am interested in floors and therefore in the compression test, as I find that when you take a portland cement of the finer grinding than that of standard portland cement, the more fine sand you add to the mix to take up the difference of the standard residue of 22 per cent.

The Effect of Varied Curing Conditions Upon the Compressive Strength of Mortars and Concretes.

By Herbert J. Gilkey.*

Introduction.

The curing conditions considered are only those dependent upon the effect of moisture, or the lack of it, on the strength of portland cement mortars and concretes. Other important factors, such as temperature changes, impure curing waters, etc., are not here considered.

To the thoughtful concrete operator, designer, or investigator, many questions as to the effect of curing conditions upon the strength of concrete must continually occur. Some of these may be listed as follows:

1. For how long should concrete be kept moist after being placed?
2. Is intermittent wetting or sprinkling as effective as genuine moist curing?
3. Is an early drying out permanently detrimental to the concrete? (a) If the concrete be subsequently subjected to standard moist conditions? (b) If the concrete remain in the dried out state?
4. Will alternate saturation and drying out give a concrete stronger or weaker than one subjected to one condition or the other?
5. Concrete cured moist is stronger than that dry cured, but is the difference enough to be of practical interest?
6. Concrete cured moist but dried out at test is stronger than the same concrete tested wet, why, and to what extent?
7. Is moist storage as essential to wet mixed concrete as it is to dryer mixes?
8. Do mixes of different richness respond in the same way and to the same extent, to different curing methods?
9. Is there any essential difference between the curing of concrete and mortars?
10. How is curing affected by size of specimen or body of concrete?
11. If one of a series of specimens has been left exposed to the air a day or more longer than its mates (possibly for recapping, as sometimes occurs) will this specimen differ in strength due to the difference in treatment? (a) If the mold were left on during exposure? (b) If the mold were removed and the specimen in contact with the air during the exposure?

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12. In building construction, would leaving forms on for longer periods assist in curing?

13. Should special precautions be taken in building construction to avoid drying out, especially if heat be used to prevent freezing?

14. Some laboratories habitually remove specimens from moist storage one day before test. Will this procedure permit a fair measure of relative strengths of: (a) Specimens of different sizes (as the standard 6 x 12-in.; 8 x 16-in., and 2 x 4-in. cylinders). (b) Specimens tested at different seasons of the year or at different locations where evaporation rates vary?

15. Is the strength change proportional to the change in moisture content: (a) If the moisture be lost before moist storage starts? (b) If the moisture be lost after moist storage but prior to test? (c) If equal moisture losses in either of the above cases be attained at different rates, i. e., if a specimen attain a given loss (1) at a rapid rate, or (2) at a slow rate?

16. How do rates of evaporation compare with rates of absorption: (a) For specimens of the same size? (b) For specimens of different sizes?

17. How does the curing environment affect the phenomena of "autogenous healing"?

18. To what extent do the results from these tests accord with other similar tests?

19. Will any rule for relation between 7-day and 28-day strengths hold true for variable curing conditions?

20. Can the subject of curing be condensed into a general law or basic principle that will enable the concrete practitioner to predict with reasonable accuracy, the strength of an otherwise known concrete, under a variety of known or assumed curing conditions?

These are some of the questions to which it has been attempted to find answers by varying storage conditions for mortars and concretes. The tests include investigations of:

1. Delayed moist storage with and without the mold left on.
2. Varying periods of moist storage by removal from storage prior to test.
3. Intermittent dipping continued for varying numbers of days.
4. Intermittent soaking and drying for varying numbers of days.
5. The changes in weight due to gain and loss of water in curing.
6. The phenomena of autogenous healing as effected by the curing condition between test and re-test.

An effort has been made to co-ordinate some work of others with the results of these tests.

Method of Treatment.—The list of questions propounded is long, if not exactly imposing. Careful perusal will reveal some duplication. One phase of the problem may apply equally to the laboratory and to the field. Yet the laboratory man may miss the point of interest to him if a field setting be used, and vice versa.

Complete answers to all questions are not guaranteed. Some problems are hit rather squarely, while others receive only superficial and glancing blows. At least some light is promised on each question propounded, and it is hoped that if the light does not illumine the dark spot, it may at least serve to start profitable trains of thought, bring forth an interchange of ideas, and experience, and stimulate further experimentation to fill in doubtful gaps.

The subject matter of this paper is not technical, even though some of the curve sheets do show evidence of congestion.

A very intentional effort has been made, at the cost of considerable repetition of certain identifying data and notes, to make each table and graph (especially each graph) self-explanatory, so that its content may be studied with a minimum amount of cross-referencing and text explanation.

The text will therefore consist only of necessary amplification of graphs and tables, with an attempt to point out just where the findings have a bearing on certain of the introductory questions. The order of treatment will correspond to the numbering of the tables and figures.

Acknowledgment.—The tests were conducted in the concrete laboratory of the Department of Civil Engineering at the University of Colorado. They were only made possible by the hearty co-operation of Prof. W. C. Huntington, head of the Department of Civil Engineering, in allotting funds from a small and over-worked departmental budget, to establish and equip a thoroughly up-to-date concrete laboratory. L. E. Richardson, Alfred Kelly, Einar Lindstrom, William Eager, and Miss Edna Bretnall, all students or former students in the college of engineering, rendered valuable assistance in making specimens and tests, and in the reduction of data.

THE TESTS.

Tables.—Table 1 gives a complete index to the tests. Data and treatment of a mortar or concrete (Col. a) for any curing condition (Col. c) can be located in the tables (Col. d) or figures (Col. e). In all cases actual proportioning of materials was done by very accurate weighing. In some series, however, the weights used in designing the mixture were based upon the equivalent loose volume, and, in other cases, upon the equivalent absolute volume. Still other series were designed by weight proportions. Moreover, the weight per cu. ft., as determined by the A. S. T. M. standard method (C. 29-21) differs from actual loose weight (untamped). The latter is, of course, too variable to use as standard loose weight, but was, nevertheless, a fairer basis for approximating commercial proportions than any other. Due to these considerations, Table 1 contains four different columns of proportions. Some men habitually think of concrete in terms of one method of proportioning, and some in another. In using the table, one may think of and compare mixtures in his own terms.

Between 600 and 700 compressive tests were made. Some series were quite comprehensive, while others were small, being outline or reconnois-

sance tests. Complete data are given for all the major series. The results from some of the smaller series appear herein only on the graph sheets. The results in both major and minor series were extremely uniform, and it is felt that the degree of dependability is very high. In most cases plotted tests follow the general trend so perfectly that even a skeleton

TABLE 1.—OUTLINE OF TESTS—INDEX AND SUMMARY.

	Series	Description	Tables No.	Fig. No.	Proportions: Loose Volume[1] Actual	Proportions: Loose Volume[1] Std.	Proportions: Wt.	Proportions: Abs. vol.	W/C	Tests made	No. spec.	No. weighings
a	Col. b	c	d	e	f	g	h	i	j	k	l	m
Mortars	M1 (a)	Delayed Storage	5, 6	1, 6, 7, 9, 10, 11, 13	1:2.26	1:2.15	1:2.5	1:2.96	0.70	1925 Sum.	84	252
	(b)	Out Before Test	5, 7	1, 6, 8, 9, 10. 11, 13	"	"	"	"	"	"	144	432
	(c)	Intermittent Wetting	5, 8	2, 7, 8, 13	"	"	"	"	"	"	72	650
	(d)	7 da. Tests (a) (b) (c)	5, 6, 7, 8	8, 9, 10, 13	"	"	"	.	"	"	50	200
	(e)	3 mo. Tests (a) (b) (c)	5,6,7,8,9	8, 9, 10, 12	"	"	"	"	"	"	20	90
	(f)	Autog. Heal 7 da.–3 mo.	5, 9	12	"	"	"	"	"	"	11	.
	M2 (a)	Molds On vs. Off—Delay	....	11	1:1.53	1:1.45	1:1.69	1:2	0.65	1925 Spr.	30	. .
	(b)	Out Before Test	....	11	"	"	"	"	"	"	12	...
	M3 (a)	Strength vs. Age	10	13	1:2.26	1:2.15	1:2.5	1:2.96	0.70	1925 Sum.	75	
Concretes	C1 (a)	Delayed Storage	11	3, 6, 7, 11	1:2:4	1:1.90:3.57	1:2.22:3.46	1:2.62:4.24	1.10	1925 Fall	24	672
	(b)	Out Before Test	11	3, 6, 8, 11	1:2:4	"	"	"	"	"	24	"
	C2 (a)	Delayed Storage	11	4, 6, 7, 11	"	"	"	"	0.80	"	24	"
	(b)	Out Before Test	11	4, 6, 8, 11	"	"	"	"	"	"	24	"
	C3 (a)	Molds On vs. Off—Delay	....	5, 6, 7, 11	1:1.53:2.83	1:1.45:2.53	1:1.69:2.45	1:2:3	0.65	1925 Spr.	22	150
	Abrams (a)	Out Before Test	....	12	1:4 (Graded from 0–1½)			....	0.66 1.09	Lewis Inst., Bull. 2 Table IV		...
	Abrams (b)	Out Before Test	....	11	1:4 (Graded from 0–1½)			....	0.82	Lewis Inst. Bull. 11, Table V		..
	Green	Out Before Test	....	11	1:2:4	....		....	....	Proc. A.S.T.M 1919 Part II p. 608		...

[1] Loose vol. actual = proportions as based on unit wt. of sand and stone shovelled loose into measure.
Loose vol. std. = proportions as based on unit wt. of sand and stone per A.S.T.M. Std. C29–21.
Cement assumed to weigh 94 lb. per cu. ft. in both cases. All tests compression. Made at 28-da. unless otherwise noted.

series gives evidence of real value. A large number of weighings (Col. m) was taken in order fully to control the weight changes.

In Table 2 are listed the general properties and characteristics of the mortars and concretes. Weights in pounds per cu. ft. and voids data are given in some detail. It is felt that these are terms in which the "concrete" public will do well to acquire the habit of thinking. In mixtures as wet as those generally used, the voids in the concrete or mortar as made

are nearly all filled with water. Thus the water-cement ratio and voids cement ratio are nearly identical if the cement and voids be expressed ir

TABLE 2.—PROPERTIES OF THE MORTARS AND CONCRETES.

	Series	Prop. loose vol. (actual)	W/C	Slump	Str. 28 da. std. spec. lb. per sq. in.	Wt. (lb. per cu. ft.) As made	At 1 da.	Saturated	Air dried	R'ge	Voids as made H_2O	Air	Total	Range	Mortar voids Vm	Voids-Cement ratio Absolute	Bulk	No. spec. of kind	Mean variation in str. (per cent)
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t
Mortars	M1 (a–f)	1:2.26	0.70	......	4200	.....	141.3	143.6	133.3 I 131.6 II	±0.5	......	......	......	......	.....	.....	.....	4	3.1
	M2 (a) (b)[1]	1:1.53	0.65	7.0	4753[1]	136.2	138.1	140.1	....	±0.5	0.3010	0.0320	0.3330	......	0.333	1.481	0.707	3[1]	2.5
	" " " "	"	"	......	4920	.....	....	.....	....	....	...	...	.....	......	..	.	..	1 to 5	4.9
	M3 (a)	1:2.26	0.70	......	4556	.....	139.9	144.0	....	.....	....	...	......	....	.....	...	..	12 to 13	4.7
Concretes	C1 (a) (b)	1:2:4	1.10	8.0	1500	143.2	144.2	145.0	138.6	±0.5	0.2265	0.0021	0.2285	±0.013	0.392	2.331	1.110	3	8.6
	C2 (a) (b)	"	0.80	1.5	3400	147.2	147.8	148.8	142.8	±0.4	0.1741	0.0099	0.1840	±0.008	0.329	1.770	0.844	3	3.9
	C3 (a)	1:1.53:2.83	0.65	1.0	4290	147.3	147.9	148.8	144.3	±0.5	0.1820	0.0130	0.1950	....	0.326	1.451	0.694	1 to 3	1.3
	Abrams (a)	1:4	0.66	......	4980	4 month strength													
			1.09	......	3640														
	" (b)	"	0.82	......	2580														
	Green	1:2:4	...	....	1466														

All mortar spec. are 2 by 4 in. except as noted. [1] 6 by 12 in. spec. made with series C3 (a).

the same units of volume. This is shown in Col. (l) to (r) inclusive. [For more detail relative to voids and voids-cement ratio, see Bulletin 137 of the University of Illinois Engineering Experiment Station.]

Table 3 gives the properties of the materials used, which were identical throughout the tests. One-size coarse aggregate (¾ to 1) was employed in order to eliminate possible variation in results due to non-uniform grading. The stone is a very silicious hard clean crushed sandstone, and from these and numerous other tests it has been proved to be an admirable aggregate for concrete. The sand was not unlike most river sands, and had been washed. Quartz particles predominated, with a fair proportion of granite and other kinds of pebbles.

TABLE 3.—PROPERTIES OF THE MATERIALS.

Material	Kind	Size	Specific Gravity	Absorption, per cent by wt.	Unit wt. (loose) lb. per cu. ft. (shovelled in)	Unit wt. (std.) A.S.T.M. C29–21	Density A.S.T.M. C29–21	Voids	Fineness Modulus
Coarse Aggr...	White Sandstone (Quartzite)..	¾–1	2.57	0.91	81.4	91.0	0.568	0.432	8.00
Fine Aggr......	Washed Gravel..............	0–4	2.66	1.30	104.0	109.5	0.660	0.340	2.67
Cement........	Colorado Portland U. S. Brand	—	3.15	Assumed		94.0	0.478	0.522	—

Table 4 requires no explanation. It represents an average of many sieve analyses.

TABLE 4.—SIEVE ANALYSIS OF SAND.

Tyler Sieve No.	4	8	14	28	48	100	Pan	Fineness Modulus
Retained on Sieve (percentage by wt.).............	0.00	10.64	19.50	21.60	27.75	15.85	4.66	—
" " " (cumulative percentage by wt.)..	0.00	10.64	30.14	51.74	79.49	95.34	—	2.67

Table 5 should be of great assistance in studying, visualizing, and evaluating the data of the major series. All the specimens from Series M1 (a)-(f) inc., are here arranged in their order of strength. Col. (a) therefore represents both the line number and the relative strength of any one specimen of these series, as compared to the strength of all the others. Since it may often be desired to know just where a specimen stood in relation to others of its own age or its own series, Cols. (b) to (h) inc., supply the data for almost any sort of comparison that could be desired. In like manner relative strengths on the basis of various percentages are shown in Cols. (n) to (s) inc. Some persons think in terms of 28-day strength, others in 7-day or 3-mo. strengths. The data of these columns give, as in the preceding case, an opportunity to compare strengths on any one of a wide range of standards. Throughout these series dry air storage was of two kinds Col. (k). I represents the air of the concrete laboratory, located in the basement with several tanks of water about. II is the air of the materials testing laboratory, upstairs and much drier. The rate of

TABLE 5.—MORTAR 2 x 4-IN. SPECIMENS ARRANGED IN ORDER OF COMPRESSIVE STRENGTH: SERIES M 1 (abcdef).

Proportions; 1:2.5 by Wt. $\frac{W}{C}=0.70$. Tested at ages of 7 da., 28 da., and 3 mo. Storage conditions: wet, dry I & II, delayed, out before test, intermittent wetting, autogenous or retest.

Order of Strength: All Spec.	All 3 mo. Spec.	All 28 da. Spec.	All 7 da. Spec.	All Delayed Spec.	All Out before test Spec.	All Intermittent Spec.	All Autogenous Spec.	Age at test (Da. or Mo.)	Storage conditions: Description (Autogenous spec. tested at 7 da. and 3 mo.)	Storage I, II, Wet	Tested Dry or Wet	Strength, lb. per sq. in.	Percentage Str. in terms of 28 Day Std. 4200	28 Day Max. 5620 (line 2)	28 Day Min. 1830 (line 114)	7 Std. 2620	3 mo. Std. 5600	Same age Std. 2620, 7 da. 4200, 28 da. 5600, 3 mo.	Reference Table	Reference Line	No. of spec. averaged	Notes
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w
1	1	..	..	..	1	..	..	3	Out at 28 da.	II	D	6250	149.0	111.4	341.5	238.4	111.6	111.6	7	11	1	..
2	..	1	..	..	2	..	..	28	Out at 21 da.	I	D	5620	133.8	100.0	307.0	214.4	100.2	133.8	"	21	4	..
3	2	..	..	1	3	1	..	3	Std.	W	W	5600	133.3	98.8	306.0	213.7	100.0	100.0	6	1	3	..
4	..	2	..	..	4	..	..	28	Out at 21 da.	II	D	5570	132.6	99.3	304.3	212.5	99.5	132.6	7	22	4	..
5	..	3	..	..	5	..	..	28	Out at 24 da.	II	D	5510	131.2	98.2	301.0	210.3	98.4	131.2	7	18	3	..
6	3	..	..	2	..	..	..	3	In at 2 da.	I	W	5460	130.0	97.3	298.3	208.2	97.5	97.5	9	15	1	..
7	..	4	..	..	6	..	..	28	Out at 24 da	I	D	5410	128.7	96.4	295.6	206.4	96.6	128.7	7	17	4	..
8	..	5	..	..	..	2	..	28	In 1 out 1 for 24	II	D	5350	127.3	95.3	292.2	204.0	95.5	127.3	8	4	4	..
9	..	6	..	..	7	..	..	28	Out at 18 da.	I	D	5350	127.3	95.3	292.2	204.0	95.5	127.3	7	23	4	..
10	..	7	..	..	8	..	..	28	Out at 26 da.	II	D	5340	127.1	95.2	291.8	203.9	95.4	127.1	7	15	4	..
11	..	8	..	..	9	..	..	28	Out at 15 da.	I	D	5330	126.8	95.1	291.2	203.3	95.3	126.8	"	24	4	..
12	..	9	..	..	10	..	..	28	Out at 22 da.	I	D	5330	126.8	95.1	291.2	203.3	95.3	126.8	"	20	4	..
13	..	10	..	..	11	..	..	28	Out at 23 da.	I	D	5310	126.4	94.6	290.0	202.6	94.8	126.4	"	19	4	..
14	..	11	..	..	12	..	..	28	Out at 26 da.	I	D	5190	123.5	92.5	283.5	198.0	92.7	123.5	"	14	4	..
15	..	12	..	..	..	3	..	28	In 2 out 2 for 23	II	D	5175	123.2	92.3	282.8	197.4	92.5	123.2	8	2	3	..
16	..	13	..	..	13	..	..	28	Out at 25 da.	I	D	5160	122.8	91.9	282.0	196.9	92.1	122.8	7	16	4	..
17	..	14	..	..	14	..	..	28	Out at 27 da.	II	D	5130	122.1	91.4	280.3	195.7	91.6	122.1	"	13	4	..
18	4	..	..	3	..	..	..	3	In at 5 da.	II	W	5120	121.8	91.2	279.8	195.3	91.4	91.4	6	9	1	..
19	5	..	..	4	15	4	1	3	Aut. wet 7, wet	W	W	5105	121.5	91.0	278.9	194.8	91.2	91.2	9	5	2	..
20	..	15	..	..	16	..	..	28	Out at 12 da.	I	D	5070	120.7	90.3	277.0	193.4	90.5	120.7	7	25	4	..
21	6	..	..	5	..	..	..	3	In at 2 da.	II	W	5060	120.4	90.2	276.4	193.0	90.4	90.4	6	3	1	..
22	..	16	..	..	17	..	..	28	Out at 27 da.	I	D	4890	116.4	87.2	267.0	186.6	87.4	116.4	7	12	4	..
23	..	17	..	..	18	..	..	28	Out 12 hr.	II	D	4820	114.7	85.8	263.3	183.9	86.0	114.7	"	9	4	..
24	7	..	..	6	..	..	..	3	In at 10 da.	II	W	4780	114.0	85.3	261.0	182.4	85.5	85.5	6	16	1	..
25	..	18	..	..	19	..	..	28	Out 12 hr.	I	D	4640	110.5	82.6	253.5	177.0	82.8	110.5	7	8	4	..
26	..	19	..	..	20	..	..	28	Out at 9 da.	I	D	4610	109.7	82.1	251.9	175.9	82.3	109.7	"	26	4	..
27	..	20	..	..	21	..	..	28	Out 7 hr.	II	D	4600	109.5	82.0	251.3	175.5	82.2	109.5	"	7	4	..
28	..	21	..	..	22	..	..	28	Out at 8 da.	I	D	4565	108.6	81.3	249.4	174.2	81.5	108.6	"	27	3	..
29	..	22	..	..	23	..	..	28	Out 1.25 hr.	II	D	4490	106.8	80.1	245.3	171.3	80.3	106.8	"	3	4	..
30	..	23	..	..	24	..	..	28	Out 7 hr.	I	D	4390	104.5	78.3	233.8	167.5	78.5	104.5	"	6	4	..
31	8	..	..	..	25	..	..	3	Out at 7 da.	II	D	4360	103.8	77.6	238.2	166.4	77.8	77.8	"	29	1	..
32	..	24	..	..	26	..	..	28	Out 2.25 hr.	II	D	4310	102.6	76.7	235.5	164.5	76.9	102.6	"	5	4	..
33	..	25	..	..	27	..	..	28	Out 2.25 hr.	I	D	4270	101.6	76.1	233.3	162.9	76.3	101.6	"	4	4	..
34	..	26	..	..	28	..	..	28	Out at 7 da.	I	D	4210	100.2	75.0	230.0	160.6	75.2	100.2	"	28	4	..
35	..	27	..	7	29	5	..	28	Std.	W	W	4200	100.0	74.8	229.3	160.2	75.0	100.0	6	1	12	..
36	..	28	..	..	30	..	..	28	Out 1.25 hr.	I	D	4150	98.8	74.0	226.7	158.4	74.2	98.8	7	2	4	..
37	9	..	..	8	..	..	..	3	In at 28 da.	I	W	4130	98.3	73.6	225.7	157.6	73.8	73.7	6	25	1	..
38	..	29	..	9	..	..	..	28	In at 2 da.	I	W	4130	98.2	73.6	225.7	157.6	73.8	98.2	"	2	4	..
39	..	30	..	..	..	6	..	28	In 1 out 1 for 6	IIW	W	4123	98.2	73.4	225.2	157.4	73.6	98.2	8	7	4	..
40	..	31	..	..	..	7	..	28	In 1 out 1 for 6	II	D	4100	97.6	73.0	224.0	156.5	73.2	97.6	"	6	4	..
41	10	..	..	10	..	..	..	3	In at 28 da.	II	W	4090	97.4	72.9	223.4	156.0	73.1	73.1	6	26	1	..
42	..	32	..	11	..	..	..	28	In at 2 da.	II	W	4040	96.2	72.0	220.8	154.1	72.2	96.2	"	3	8	..
43	11	..	..	12	..	..	2	3	Aut. Dry 7, Wet	I	W	4030	96.0	71.8	220.0	153.8	72.0	71.9	9	8	2	..
44	12	..	..	13	..	..	3	3	" " " "	II	W	4020	95.7	71.6	219.6	153.4	71.8	71.8	"	9	1	..
45	..	33	..	..	31	..	..	28	Out at 7 da.	II	D	4015	95.6	71.4	219.3	153.2	71.6	95.6	7	29	4	..
46	..	34	..	..	32	..	..	28	Out at 6 da.	I	D	3820	90.9	68.0	208.8	145.7	68.2	90.9	"	30	4	..
47	..	35	..	14	..	..	..	28	In at 3.	I	W	3805	90.6	67.8	207.9	145.2	68.0	90.6	6	4	4	..
48	..	36	..	..	33	..	..	28	Out at 5 da.	I	D	3715	88.5	66.1	203.0	141.7	66.3	88.5	7	31	5	..
49	..	37	..	15	..	..	..	28	In at 4 da	I	W	3665	87.2	65 3	200 2	139.8	65.5	87.2	6	6	4	..
50	..	38	..	..	34	..	..	28	Out at 4 da.	II	D	3610	86.0	64.3	197.2	137 8	64.5	86.0	7	34	1	..
51	..	39	..	16	..	..	..	28	In at 5 da.	I	W	3575	85.2	63.6	195 3	136.4	63.8	85.2	6	8	4	..
52	..	40	..	17	..	..	..	28	In at 6 da.	I	W	3500	83.3	62.3	191.2	133.6	62.5	83 3	"	10	4	..
53	..	41	..	18	..	..	..	28	In at 5 da.	II	W	3420	81.4	60.9	186.8	130.5	61.1	81.4	"	9	4	..
54	..	42	..	..	35	..	..	28	Out at 4 da.	I	D	3418	81.4	60.9	186.7	130.4	61.0	81.4	7	33	4	..
55	13	..	..	..	36	..	..	3	Out at 2 da.	II	D	3360	80.0	59.9	183.5	127.3	60.0	60.0	"	38	1	..
56	.	43	..	19	..	..	..	28	In at 9 da.	I	W	3310	78.8	59.0	180.8	126.3	59.1	78.8	6	15	4	..
57	14	..	..	..	37	..	4	3	Aut. Wet 7, Dry	I	D	3310	78.8	59.0	180.8	126.3	59.1	59.1	9	4	1	..
58	..	44	..	20	..	..	..	28	In at 7 da.	I	W	3260	77.6	58.1	178.0	124.4	58.2	77.6	6	12	4	..

TABLE 5.—MORTAR 2 x 4-IN. SPECIMENS ARRANGED IN ORDER OF COMPRESSIVE STRENGTH: SERIES M 1 (abcdef). (*Continued.*)

Proportions; 1:2.5 by Wt. $\frac{W}{C}=0.70$. Tested at ages of 7 da., 28 da., and 3 mo. Storage conditions: wet, dry I & II, delayed, out before test, intermittent wetting, autogenous or retest.

Order of Strength: All Spec.	All 3 mo. Spec.	All 28 da. Spec.	All 7 da. Spec.	All Delayed Spec.	All Out before test Spec.	All Intermittent Spec.	All Autogenous Spec	Age at test (Da. or Mo.)	Storage conditions: Description (Autogenous spec. tested at 7 da. and 3 mo.)	Storage: I, II, Wet	Tested: Dry or Wet	Strength, lb. per sq. in.	Percentage Str. in terms of: 28 Day Std. 4200	28 Day Max. 5620 (line 2)	28 Day Min. 1830 (line 114)	7 Std. 2620	3 mo Std. 5600	Same age Std. 2620, 7 da. 4200, 28 da. 5600, 3 mo.	Reference: Table	Reference: Line	No. of spec. averaged	Notes
a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w
59	..	..	1	..	38	..	..	7	Out at 4 da..........	I	D	3240	77.2	57.7	177.0	123.6	57.8	123.6	7	35	1	..
60	..	..	2	..	39	..	..	7	Out at 3 da..........	II	D	3150	75.0	56.1	172.1	120.2	56.2	120.2	"	34	1	..
61	..	45	..	..	40	..	..	28	Out at 3 da..........	I	D	3115	74.1	55.6	170.2	118.8	55.6	74.1	"	35	2	..
62	..	46	..	21	..	..	..	28	In at 12 da..........	I	W	3100	73.8	55.3	169.3	118.3	55.4	73.8	6	17	4	..
63	..	..	3	..	41	..	..	7	Out at 5 da..........	II	D	3100	73.8	55.8	169.3	118.3	55.4	118.3	7	38	1	..
64	..	47	..	..	..	8	..	28	Dip 2–24 da. incl	II	D	3060	72.8	54.5	167.2	116.8	54.6	72.8	8	8	4	..
65	..	48	..	22	..	..	..	28	In at 15 da..........	I	W	3000	71.4	53.5	163.8	114.5	53.6	71.4	6	18	4	..
66	..	..	4	..	42	..	..	7	Out at 4 da..........	II	D	2940	70.0	52.4	160.6	112.2	52.5	112.2	7	36	1	..
67	..	49	..	23	..	..	..	28	In at 18 da..........	I	W	2900	69.1	51.7	158.4	110.6	51.8	69.1	6	19	4	..
68	..	50	..	..	43	..	..	28	Out at 3 da..........	II	D	2896	69.0	51.5	158.2	110.5	51.6	69.0	7	36	2	..
69	..	..	5	..	..	9	..	7	In 1 out 1 for 6......	I	D	2870	68.3	51.1	156.7	109.5	51.2	109.5	8	5	1	..
70	..	..	6	..	44	..	..	7	Out 7 hr............	II	D	2820	67.2	50.3	154.0	107.6	50.4	107.6	7	7	1	..
71	..	..	7	..	45	..	..	7	Out 12 hr...........	II	D	2760	65.8	49.2	150.7	105.3	49.3	105.4	"	9	1	..
72	..	..	8	..	..	10	..	7	In 1 out 1 for 6......	II	D	2760	65.7	49.2	150.7	105.3	49.3	105.4	8	6	1	..
73	..	..	9	..	46	..	..	7	Out 7 hr............	I	D	2700	64.3	48.1	147.5	103.0	48.2	103.0	7	6	1	..
74	..	51	..	24	..	..	..	28	In at 21 da..........	I	W	2690	64.1	47.9	146.9	102.6	48.0	64.1	6	20	4	..
75	..	..	10	..	47	..	..	7	Out 1.25 hr..........	II	D	2690	64.1	47.9	146.9	102.6	48.0	102.6	7	3	1	..
76	..	..	11	..	48	..	..	7	Out at 2 da..........	II	D	2670	63.5	47.6	145.8	101.9	47.7	101.9	"	32	1	..
77	..	52	..	..	49	..	..	28	Out at 2 da..........	I	D	2667	63.3	47.5	145.6	101.8	47.6	63.3	"	37	4	..
78	..	..	12	25	50	11	5	7	Aut. Wet 7, Dry.....	W	W	2630	62.6	46.9	143.6	100.4	47.0	100.4	9	4	1	..
79	..	..	13	26	51	12	..	7	Std.................	W	W	2620	62.4	46.7	143.1	100.0	46.8	100.0	6	1	3	..
80	..	53	..	..	52	..	..	28	Out at 2 da..........	II	D	2620	62.4	46.7	143.1	100.0	46.8	62.4	7	38	1	..
81	..	..	14	27	53	13	6	7	Aut. Wet 7, Wet.....	W	W	2615	62.3	46.6	142.8	99.8	46.7	99.8	9	5	2	..
82	..	..	15	..	54	..	..	7	Out 2.25 hr..........	II	D	2590	61.7	46.2	141.5	98.8	46.3	98.8	7	5	1	..
83	..	54	..	..	..	14	..	28	Dip 2–7 da. incl	II	D	2560	61.0	45.6	139.8	97.7	45.7	61.0	8	9	4	..
84	..	55	..	28	..	..	..	28	In at 24 da..........	I	W	2440	58.1	43.5	133.3	93.1	43.6	58.1	6	21	4	..
85	..	..	16	29	..	..	..	7	In at 2 da...........	I	W	2430	57.8	43.3	132.8	92.8	43.4	92.8	"	2	2	..
86	..	..	17	..	..	15	..	7	Dip 2–3 da. incl......	II	D	2400	57.1	42.8	131.1	91.6	42.9	91.6	8	11	1	..
87	15	..	..	30	55	16	..	3	Dry.................	II	D	2388	56.9	42.5	130.5	91.1	42.6	42.6	6	28	5	..
88	..	56	..	31	56	17	..	28	"	I	D	2350	56.0	41.9	128.4	89.7	42.0	56.0	"	13	4	1
89	..	57	..	..	..	18	..	28	Dip 2–3 da. incl......	I	D	2340	55.7	41.7	127.9	89.3	41.8	55 7	8	12	4	..
90	..	..	18	32	..	..	..	7	In at 2 da...........	II	W	2293	54.6	40.8	125.3	87.5	40.9	87.5	6	3	3	..
91	16	..	..	33	57	19	..	3	Dry.................	I	D	2278	54.2	40.5	124.4	87.3	40.6	40.7	"	13	5	..
92	..	58	..	..	..	20	..	28	Dip 2–3 da. incl......	II	D	2253	53.6	40.1	123.2	86.0	40.2	53.6	8	11	4	..
93	..	..	19	34	..	..	..	7	In at 3 da...........	II	W	2245	53.4	40.0	122.6	85.7	40.1	85.7	6	5	2	..
94	..	..	20	35	..	..	..	7	In at 3 da...........	I	W	2230	53.1	39.7	121.8	85.1	39.8	85.1	6	4	2	..
95	..	..	21	..	..	21	..	7	Dip 2–7 da. incl......	II	D	2220	52.8	39.6	121.3	84.7	39.7	84.7	7	9	1	..
96	..	59	..	36	..	..	..	28	In at 25 da..........	I	W	2215	52.7	39.4	121.0	84.5	39.5	52.7	6	22	4	..
97	..	..	22	..	..	22	..	7	Dip 2–7 da. incl......	I	D	2200	52.4	39.2	120.2	84.0	39.3	84.0	8	10	1	..
98	..	60	..	37	..	..	..	28	In at 26 da..........	I	W	2150	51.2	38.3	117.5	82.0	38.4	51.2	6	23	4	..
99	..	..	23	..	..	23	..	7	In 2 out 2 for 7......	II	W	2150	51.2	38.3	117.5	82.0	38.4	82.0	8	3	1	..
100	..	..	24	38	..	..	..	7	In at 4 da...........	II	W	2090	49.7	37.2	114.2	79.7	37.3	79 7	6	7	1	..
101	..	..	25	39	..	..	..	7	" " " "	I	W	2065	49.2	36.8	112.8	78.8	36 9	78.8	6	6	2	..
102	..	..	26	..	58	..	..	7	Out 1.25 hr..........	I	D	2030	48.7	36.1	110.9	77.5	36.2	75.5	7	2	1	..
103	..	61	..	40	59	24	..	28	Dry.................	II	D	1996	47.5	35.6	109.1	76.2	35.7	47.5	6	28	12	..
104	17	..	..	41	60	25	7	3	Aut. Dry 7, Dry.....	II-I	D	1935	46.1	34.5	105.7	73.8	34.6	34.6	9	7	2	..
105	18	..	..	42	61	26	8	3	" " " "	I-I	D	1930	45.9	34.4	105.5	73.6	34.5	34.5	"	6	2	..
106	..	..	27	43	62	27	9	7	" " " "	I	D	1930	45.9	34.4	105.5	73.6	34.5	73.6	"	"	2	..
107	..	..	28	44	..	..	..	7	In at 5 da...........	II	W	1920	45.7	34.2	104.8	73.3	34.3	73.3	6	9	1	..
108	..	.	29	45	63	28	10	7	Aut. Dry 7, Dry.....	II	D	1915	45.6	34.1	104.6	73 1	34.2	73.1	9	7	2	..
109	..	..	30	46	..	..	..	7	In at 6 da...........	II	W	1900	45.2	33 8	103.8	72.5	33.9	72.5	6	11	1	..
110	..	..	31	47	64	29	..	7	Dry.................	II	D	1875	44.7	33.4	102.5	71.6	33.5	71.6	"	14	4	..
111	..	..	32	48	65	30	..	7	"	I	D	1871	44.5	33 3	102 2	71.4	33.4	71 4	"	12	4	..
112	..	..	33	49	66	31	11	7	Aut. Dry 7, Wet.....	II	D	1850	44.0	32.9	101.1	70.6	33.0	70.6	9	9	1	..
113	..	62	..	50	..	..	..	28	In at 27 da..........	I	W	1850	44.0	32.9	101.1	70.6	33.0	44.0	6	24	4	..
114	..	63	..	51	67	32	..	28	Dry.................	II	D	1830	43.6	32.6	100.0	69.8	32.7	43.6	"	26	4	1
115	..	..	34	52	..	..	..	7	In at 5 da...........	I	W	1815	43.2	32.3	99.2	69.3	32.4	69.3	"	8	2	..
116	..	..	35	53	68	33	12	7	Aut. Dry 7, Wet.....	I	D	1810	43.1	32.2	98.9	69.1	32.3	69.1	9	8	2	..
117	..	..	36	54	..	..	..	7	In at 6 da...........	I	W	1650	39.3	29.4	90.2	63.0	29.5	63.0	6	10	1	..

[1] Line 88. Doubtful. Other tests give 1838 as mean strength. Both are the average of 4 specimens that depart less than 5 per cent from mean. If 1838 be correct, this spec. would lie between lines No. 113 and 114.

evaporation II was from 1.5 to 4 times that of I, as determined by cans of water, of equal exposed surfaces, used as evaporation controls. Autographic temperature and humidity records would have been desirable, but we had not the equipment for obtaining these. The numeral I or II designates only the air in which the specimen was stored when not in water.

TABLE 6.—DELAYED WET STORAGE OF MORTARS: SERIES—M 1 (a).

Summary and comparison of 7, 28, and 84 da. (approx. 3 mo.) compr. tests of 2 x 4-in. mortar spec. Prop., 1:25 by wt. $\frac{W}{C}=0.70$. All spec. removed from mold at 1 day. Subsequent treatment (col. "c") as follows: Wet = stored in water upon removal from mold (Std. condition); I = stored in air of Concr. Lab. until immersed; II = stored in air of Mat. Test. Lab. until immersed. (Air much drier than that of Concr. Lab.). In col. "d"; W = tested wet, D = tested dry. All spec. tested wet except those never placed in water (limiting case). Certain spec., i.e. the "Drys," are repeated to facilitate comparison. 7 and 84 da. data are meagre. These are skeleton series and should be considered tentative.

Line No.	Storage Conditions				Strengths lb. per sq. in.			Strength Ratios (per cent)						No. of Tests Averaged		
	Immersed Age (da.)	In Air (da.)	Storage (kind)	Condition at test				Std. spec. same age			Std. 28-da. spec.					
					7	28	84	7	28	84	7	28	84	7	28	84
	Column a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p
1	Std. 1	0	Wet	W	2620	4200	5600	100.0	100.0	100.0	62.4	100.0	133.3	3	12	3
2	2	1	I	"	2430	4130	5460	92.8	98.2	97.5	57.8	98.2	130.0	2	4	1
3	2	1	II	"	2293	4040	5060	87.5	96.2	90.4	54.6	96.2	120.4	3	8	1
4	3	2	I	"	2230	3805	.....	85.1	90.6	.....	53.1	90.6	.....	2	4	..
5	3	2	II	"	2245	.....	.....	85.7	......	.....	53.4	......	.....	2	..	..
6	4	3	I	"	2065	3665	.....	78.8	87.2	.....	49.2	87.2	.....	2	4	..
7	4	3	II	"	2090	.....	.....	79.7	......	.....	49.7	......	.....	1	..	..
8	5	4	I	"	1815	3575	.....	69.3	85.2	.....	43.2	85.2	.....	2	4	..
9	5	4	II	"	1920	3420	5120	73.3	81.4	91.4	45.7	81.4	121.8	1	4	1
10	6	5	I	"	1650	3500	.....	63.0	83.3	.....	39.3	83.3	.....	1	4	..
11	6	5	II	"	1900	.....	.....	72.5	......	.....	45.2	......	.....	1	..	..
12	Dry, 7	6	I	D. W.	1871	3260[1]	.....	71.4	77.6	.....	44.5	77.6	.....	4	4	..
13	Dry	6, 27, 83	I	D	1871	2350	2278	71.4	56.0[1]	40.7	44.5	56.0[1]	54.2	4	4	5
14	"	6, 27, 83	II	D	1875	1996	2388	71.6	47.5	42.6	44.7	47.5	56.9	4	12	5
15	9	8	I	W	.....	3310	.....	......	78.8	.....	.....	78.8	.....	..	4	..
16	10	9	II	"	.....	.....	4780	......	......	85.4	.....	......	114.0	..	..	1
17	12	11	I	"	.....	3100	.....	......	73.8	.....	.....	73.8	.....	..	4	..
18	15	14	I	"	.....	3000	.....	......	71.4	.....	.....	71.4	.....	..	4	..
19	18	17	I	"	.....	2900	.....	......	69.1	.....	.....	69.1	.....	..	4	..
20	21	20	I	"	.....	2690	.....	......	64.1	.....	.....	64.1	.....	..	4	..
21	24	23	I	"	.....	2440	.....	......	58.1	.....	.....	58.1	.....	..	4	..
22	25	24	I	"	.....	2215	.....	......	52.7	.....	.....	52.7	.....	..	4	..
23	26	25	I	"	.....	2150	.....	......	51.2	.....	.....	51.2	.....	..	4	..
24	27	26	I	"	.....	1850	.....	......	44.0	.....	.....	44.0	.....	..	4	..
25	Dry, 28	27	I	D. W.	.....	2350[1]	4130	......	56.0[1]	73.7	.....	56.0[1]	98.3	..	4	1
26	" 28	27	II	D. W.	.....	1830	4090	......	43.6	73.1	.....	43.6	97.4	..	4	1
27	Dry	6, 27, 83	I	D	1871	2350[1]	2278	71.4	56.0[1]	40.7	44.5	56.0[1]	54.2	4	4	5
28	"	6, 27, 83	II	D	1875	1996	2388	71.6	47.5	42.7	44.7	47.5	56.9	4	12	5

[1] Col. "f" Lines 12, 25, and 27. Doubtful. Other tests give 1838 as mean strength. Both are the average of 4 specimens that depart less than 5 per cent from mean.

It may have been exposed to that air for the full curing period or for only a day. Those details are covered in Col. (j). In Col. (1), "Tested" refers to whether the specimen was tested immediately upon removal from water, or allowed to dry out for some period before test. Any specimen not tested wet (immediately upon removal) is here classified as D or dry, in Col. (1).

Cols. (t) and (u) give the cross-referencing necessary to locate any specimen in Tables 6, 7, 8, or 9 from which Table 5 is compiled.

TABLE 7.—DRYING OUT BEFORE TEST. MORTARS: SERIES M 1 (b) (d) (e).

Summary and comparison of 7, 28, and 84 da. (approx. 3 mo.) compr. tests of 2 x 4-in. mortar spec. Prop. 1:2.5 by wt.; $\frac{w}{c}=0.70$. All spec. removed from mold at 1 day and stored in water (except *dry* spec. kept in air until test). Removed from water at varying ages and stored as indicated in Col. "c" until test. I=stored in air of Concr. Lab. when out of water. II=stored in air of Mat. Test. Lab. when out of water. (Air much drier than that of Concr. Lab.). Wet=standard spec. immersed until test. In Col. "d"; W=tested wet (std. spec. only). D=tested dry. Certain spec. are repeated to facilitate comparison. 7 and 84 day data are meager. These are skeleton series and should be considered tentative.

Line No.	Storage Conditions				Strengths lb. per sq. in.			Strength Ratios (per cent)						No. of Tests Averaged		
	Out of water Age (da.)	In Air (da.)	Storage (kind)	Condition at test	7	28	84	Std. spec. same age 7	Std. spec. same age 28	Std. spec. same age 84	Std. 28 da. spec. 7	Std. 28 da. spec. 28	Std. 28 da. spec. 84	7	28	84
	Column a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p
1	Std. 7, 28, 84	0. hr.	Wet	W	2620	4200	5600	100.0	100.0	100.0	62.4	100.0	133.3	3	12	3
2	" (—)	1.25 hr.	I	D	2030	4150	....	75.5	98.8	.....	48.7	98.8	.....	1	4	..
3	" "	1.25 hr.	II	D	2690	4490	....	102.6	106.8	.....	64.1	106.8	.....	1	4	..
4	" "	2.25 hr.	I	D	....	4270	....	.....	101.6	.....	....	101.6	.....	..	4	..
5	" "	2.25 hr.	II	D	2590	4310	....	98.8	102.6	.....	61.7	102.6	.....	1	4	..
6	" "	7 hr.	I	D	2700	4390	....	103.1	104.5	.....	64.3	104.5	.....	1	4	..
7	" "	7 hr.	II	D	2820	4600	....	107.6	109.5	.....	67.2	109.5	.....	1	4	..
8	" "	12 hr.	I	D	....	4640	....	.....	110.5	.....	....	110.5	.....	..	4	..
9	" "	12 hr.	II	D	2760	4820	....	105.4	114.7	.....	65.8	114.7	.....	1	4	..
10	Std. 7, 28, 84	0 da	Wet	W	2620	4200	5600	100.0	100.0	100.0	62.4	100.0	133.3	3	12	3
11	Wet, 28.......	0, 56 da.	Wet, II	W, D	....	4200	6250	.....	100.0	111.6	....	100.0	149.0	..	4	1
12	27.......	1, 56 da.	I	D	....	4890	....	.....	116.4	.....	....	116.4	.....	..	4	..
13	27.......	1 da.	II	D	....	5130	....	.....	122.1	.....	....	122.1	.....	..	4	..
14	26.......	2	I	D	....	5190	....	.....	123.5	.....	....	123.5	.....	..	4	..
15	26.......	2	II	D	....	5340	....	.....	127.1	.....	....	127.1	.....	..	4	..
16	25.......	3	I	D	....	5160	....	.....	122.8	.....	....	122.8	.....	..	4	..
17	24.......	4	I	D	....	5410	....	.....	128.7	.....	....	128.7	.....	..	4	..
18	24.......	4	II	D	....	5510	....	.....	131.2	.....	....	131.2	.....	..	3	..
19	23.......	5	I	D	....	5310	....	.....	126.4	.....	....	126.4	.....	..	4	..
20	22.......	6	I	D	....	5330	....	.....	126.8	.....	....	126.8	.....	..	4	..
21	21.......	7	I	D	....	5620	....	.....	133.8	.....	....	133.8	.....	..	4	..
22	21.......	7	II	D	....	5570	....	.....	132.6	.....	....	132.6	.....	..	4	..
23	18.......	10	I	D	....	5350	....	.....	127.3	.....	....	127.3	.....	..	4	..
24	15.......	13	I	D	....	5330	....	.....	126.8	.....	....	126.8	.....	..	3	..
25	12.......	16	I	D	....	5070	....	.....	120.7	.....	....	120.7	.....	..	4	..
26	9.......	19	I	D	....	4610	....	.....	109.7	.....	....	109.7	.....	..	4	..
27	8.......	20	I	D	....	4565	....	.....	108.6	.....	....	108.6	.....	..	3	..
28	(Std. 7), 7, —	0, 21, —	Wet, I	W. D.	2620	4210	....	100.0	100.2	.....	62.4	100.2	.....	3	4	..
29	(Std. 7), 7, 7..	0, 21, 77	Wet, II, II	W. D. D.	2620	4015	4360	100.0	95.6	77.8	62.4	95.6	103.8	3	4	1
30	6.....	22	I	D	....	3820	....	.....	90.9	.....	....	90.9	.....	..	4	..
31	5.....	23	I	D	....	3715	....	.....	88.5	.....	....	88.5	.....	..	5	..
32	2.....	5	II	D	2670	.....	....	101.9	......	.....	63.5	......	.....	1	..	..
33	4.....	24	I	D	....	3418	....	.....	81.4	.....	....	81.4	.....	..	4	..
34	3.....	4, 24	II	D	3150	3610	....	120.2	86.0	.....	75.0	86.0	.....	1	1	..
35	4.....	3, 25	I	D	3240	3115	....	123.6	74.1	.....	77.2	74.1	.....	1	2	..
36	4.....	3, 25	II	D	2940	2896	....	112.2	69.0	.....	70.0	69.0	.....	1	2	..
37	2.....	26	I	D	....	2667	....	.....	63.5	.....	....	63.5	.....	..	4	..
38	5.....	2, 26, 82	II	D	3100	2620	3360	118.4	62.4	60.0	73.8	62.4	80.0	1	1	1
39	Dry..........	6, 27, 83	I	D	1871	2350[1]	2278	71.4	56.0[1]	40.7	44.6	56.0[1]	54.2	4	4	5
40	Dry..........	6, 28, 83	II	D	1875	1996	2388	71.6	47.5	42.7	44.7	47.5	56.9	4	12	5

[1] Line 39, Col. "f." Doubtful. Other tests give 1838 as mean strength. Both are the average of 4 specimens that depart less than 5 per cent from mean.

Tables 6, 7, 8, and 9 are, as stated, the source of Table 5. They give a direct comparison of companion specimens at different ages whenever tests other than 28-day were made.

Table 9 gives a little data on specimens tested at 7 days, then re-stored in air or water, and tested again at 3 months. The existence of the phenomena of autogenous or self-healing has long been known, but there seems to be little record of effort to measure the extent of possible healing after a former test, or the reasons for, or details of the process. Abrams[1,2] has made passing mention of the fact on several occasions, and ascribes the healing to the deposition of soluble compounds in and across the minute cracks present at and after testing to ultimate strength.

TABLE 8.—INTERMITTENT WETTING MORTARS: SERIES—M 1 (c).

Summary and comparison of 7, 28, and 86 da. (Approx. 3 mo.) compr. tests of 2 x 4-in. mortar spec. Prop. 1:2.5 by wt.; $\frac{W}{C}=0.70$. All spec. removed from mold at age of 1 day. Subsequent treatment as indicated in Col. "a," "b," and "c." In Col. "b": I=stored in air of Concr. Lab. when out of water; II=stored in air of Mat. Test. Lab. when out of water. (Air much drier than that of Concr. Lab.); Wet=stored in water at 1 da. for entire period (Std. condition). Controls (Lines 1, 13, and 14) are repeated from other tables for comparison.

Line No.	Storage Conditions			Strengths, lb. per sq. in.			Strength Ratios (per cent)						No. of Tests Averaged		
							Std. spec. same age			Std. 28-da. spec.					
	Description	Storage (kind)	Condition at Test	7	28	84	7	28	84	7	28	84	7	28	84
	Column a	b	c	d	e	f	g	h	i	j	k	l	m	n	o
	Std.	II	D	2620	4200	5600	100.0	100.0	100.0	62.4	100.0	133.3	3	12	3
2	In 2, out 2 for 23 da.	II	D	....	5175	....	.....	123.2	.....	....	123.2	.....	..	3	..
3	" " 7 "	II	W	2150	.....	....	82.0	.....	.....	51.2	.....	.....	1	..	..
4	In 1, out 1 for 24 da.	II	D	....	5350	....	.....	127.3	.....	....	127.3	.....	..	4	..
5	" " 6 "	I	D	2870	.....	....	109.5	.....	.....	68.3	.....	.....	1	..	..
6	" " 6 "	II	D	2760	4100	....	105.3	97.6	.....	65.7	97.6	.....	1	4	..
7	" " 6 "	II & wet	W	....	4123	....	.....	98.2	.....	....	98.2	.....	..	4	..
8	Dip 2nd–24th da. incl.	II	D	....	3060	....	.....	72.8	.....	....	72.8	.....	..	4	..
9	" 2nd– 7th " "	II	D	2220	2560	....	84.7	61.0	.....	52.8	61.0	.....	1	4	..
10	" " " " "	I	D	2200	.....	....	84.0	.....	.....	52.4	.....	.....	1	..	..
11	" 2nd–3rd " "	II	D	2400	2253	....	91.6	53.6	.....	57.1	53.6	.....	1	4	..
12	" " " "	I	D	....	2340	....	.....	55.7	.....	....	55.7	.....	..	4	..
13	Dry	I	D	1871	2350[1]	2278	71.4	56.0[1]	40.1	44.6	56.0[1]	54.2	4	4	5
14	"	II	D	1875	1996	2388	71.6	47.5	42.7	44.7	47.5	56.9	4	12	5

[1] Col. "e" Line 13. Doubtful. Other tests give 1838 as mean strength. Both are the average of 4 specimens that depart less than 5 per cent from the mean.

It has been fully demonstrated by many unpublished data of this, as well as other laboratories, that specimens tested to the ultimate will, if load be released before severe shattering or crushing has occurred, continue to gain strength under favorable conditions of storage. In other words, the old cracks heal and the curing process continues as in an untested specimen.

These tests indicate quite conclusively that neither the healing nor the curing (except in a slight degree, see Fig. 13) occurs if the specimen be kept dry. Lines 6 and 7 of Table 9 show specimens tested at 7 days and again at 3 months, giving almost identical strengths. Lines 8 and 9 show specimens dry for the first week, then wet between tests, that more than doubled in strength after having been tested to the 7-day ultimate. Line 5

[1] Bulletin 71, Uni. of Illinois Eng'r'g Exp. Sta. [2] Test of 40 ft. Rein. Concr. High. Bri. A. S. T. M. 1913.

about doubled its 7-day strength after wet storage, for both the first week and period between, while Line 4 wet before the 7-day test but dry after, shows a gain of one-fourth its original strength.

This last example may appear to contradict the statement that au-

TABLE 9.—AUTOGENOUS HEALING OF MORTARS; ALSO 3-MO. STRENGTHS FOR VARIOUS CONDITIONS OF STORAGE: SERIES—M 1 (f) (e).

(See notes with 3 preceding tables.) Lines 1 to 3 incl. and parts of 10 to 18 incl. are repeated from other tables for comparison. Spec. on lines 4 to 9 incl. were tested dry at 7 day, re-stored as noted, and retested at 3 mo. [Other tests (not included in this paper) show that a tested specimen, if not seriously shattered, will develop a future strength approximately equal to the strength that it would have developed if not previously tested. It is sometimes difficult and often impossible to apply the maximum load to a specimen without shattering action or serious structural damage. This is especially true of strong mixtures rich in cement, low in water, or well matured.] None of these specimens was seriously injured at first test and the evidence may be considered to be about as reliable as that from original tests with same storage conditions.

Line No.	Storage Conditions				Strengths, lb. per sq. in.			Strength Ratios (per cent)						No. of Tests Averaged		
	Description		Storage (kind)	Condition at Test				Std. spec. same age			Std. 28 da. spec.					
					7	28	84	7	28	84	7	28	84	7	28	84
	Column a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p
	Regular specimens repeated for comparison with autogenous tests below															
1	Std.		Wet	W	2620	4200	5600	100.0	100.0	100.0	62.4	100.0	133.3	3	12	3
2	Dry		I	D	1871	2350[1]	2278	71.4	56.0[1]	40.7	44.6	56.0[1]	54.2	4	4	5
3	"		II	D	1875	1996	2388	71.6	47.5	42.7	44.7	47.5	56.9	4	12	5
	Autogenous Healing. Tested at 7 da. and retested at 3 mo. (87 da.)															
4	7 da. Std.	then dry	W & I	W & D	2630	.....	3310	100.4	......	59.1	62.6	......	78.8	1	..	1
5	" "	" wet	W & W	W & W	2615	.....	5105	99.8	......	91.2	62.3	......	121.5	2	..	2
6	7 da. dry	" dry	I & I	D & D	1930	.....	1930	73.6	......	34.5	45.9	......	45.9	2	..	2
7	"	" "	II & I	D & D	1915	.....	1935	73.1	......	34.6	45.6	......	46.1	2	..	2
8	"	" wet	I & W	D & W	1810	.....	4030	69.1	......	71.9	43.1	......	96.0	2	..	2
9	"	" "	II & W	D & W	1850	.....	4020	70.6	......	71.8	44.6	......	95.7	1	..	1
	Other 3-mo. tests (not autogenous) repeated for comparison															
10	28 da. dry	then wet	I	W	....	.....	4130	.....	......	73.7	....	......	98.3	..	..	1
11	"	"	II	W	....	.....	4090	.....	......	73.1	....	......	97.4	..	..	1
12	10 "	"	II	W	....	.....	4780	.....	......	85.4	....	......	114.0	..	..	1
13	5 "	"	II	W	....	.....	5120	.....	......	91.4	....	......	121.8	..	..	1
14	2 "	"	II	W	....	.....	5060	.....	......	90.4	....	......	120.4	..	..	1
15	2 "	"	I	W	....	.....	5460	.....	......	97.5	....	......	130.0	..	..	1
16	28 da. wet	then dry	II	D	....	.....	6250	.....	......	111.8	....	......	149.0	..	..	1
19	"	"	II	D	....	.....	4360	.....	......	77.8	....	......	103.8	..	..	1
18	"	"	II	D	....	.....	3360	.....	......	60.0	....	......	80.0	..	..	1

[1] Col. "e". Line 2. Doubtful. Other tests give 1838 as mean strength. Both are the average of 4 specimens that depart less than 5 per cent from mean.

togenous healing cannot take place if the specimen be kept dry. A very small part of this increase in strength may be due to the autogenous healing that went on within the specimen before it dried out sufficiently to stop further action. In the case of a 2 x 4-in. specimen this would not be great, as the drying is rapid. As other data of these tests and tests by others (Green, Abrams, etc., to be mentioned later) clearly demonstrate,

dry concrete is from 10 to 35 per cent stronger than the same concrete wet. The 26 per cent strength gain is due largely, if not entirely, to the drying out and not to autogenous healing action. This statement is borne out by short time retests. If a specimen be tested to ultimate but not seriously damaged (to the eye) an immediate retest will show that it still has a considerable load carrying capacity. This may be as high as 90 per cent of the ultimate load. If the specimen was wet at test it may be left in the air until the next day and a retest will show greater strength than the maximum of the day before. This is not a healing, but simply the 15 to 35 per cent gain due to hardening added to the 80 or 90 per cent strength that remained after maximum load was reached. It is impossible to obtain a retest on a strong mixture because of the shattering action that occurs at or very close to the ultimate strength.

With usual testing equipment and the exercise of considerable care to reverse the machine at once, when maximum load is reached, it is possible to salvage all of the 7-day test specimens and many of the 28-day specimens of concrete of usual job proportions. The writer has done this on more than one hundred 6 x 12-in. job-concrete tests. The 28-day strengths of the specimens tested at 7 days show 28-day strengths almost identical with those of the regular 28-day specimens tested for the first time at 28 days.

The specimens of Table 9 did not apparently heal 100 per cent. From the Universal Curing Chart (Fig. 13, to be discussed later) one may obtain information as to the probable strength that 3-month specimens receiving identical storage treatment but not tested at 7 days would have developed. These data are tabulated below:

Line (Table 4)		Storage	Tested	Strength (per cent of 28-Day Standard)		
				7-day Test	3-mo. Retest	3-mo. by Fig. 13
4	7 da. std., then dry	W. & I.	W. & D.	62.6	78.8	104.0
5	" " " wet	W. & W.	W. & W.	62.3	121.5	133.0
6	" dry " dry	I. & I.	D. & D.	45.9	45.9	57.0
7	" " " "	II. & I.	D. & D.	45.6	46.1	57.0
8	" " " wet	I. & W.	D. & W.	43.1	96.0	119.0
9	" " " "	II. & W.	D. & W.	44.6	95.7	119.0

A loss of about 23 per cent due to the 7-day test appears to have been sustained by 8 and 9. In these two cases, however, the 3-mo. ultimate strength was more than double the 7-day ultimate. The specimen most favorably treated for healing, No. 5, was within about 10 per cent of its normal 3-mo. strength. As stated, 4 shows some gain, probably due entirely to drying action, while 6 and 7 gained autogenously, or perhaps by additional curing of the undamaged portion, enough strength to offset the weakening caused by the original test, so that the 3-mo. strength is equal to the 7-day strength instead of 10 to 20 per cent greater as in the case of the regularly dry-cured specimen.

TABLE 10.—NORMAL INCREASE IN COMPRESSIVE STRENGTH WITH AGE: SERIES—M 3 (a).

Wet storage after one day. Tested wet. Mortar 2 x 4-in. specimens. 1:2.5 by wt. $\frac{w}{c}=0.70$. Each value is the average of test results from 12 or 13 cylinders.

Age at test (days)	2	3	5	7	28	(3 mo.) 86
Unit Compr. Str. (lb. per sq. in.)	1200	1558	2104	2414	4556	5707
Percentage of 7 da. str.	49.7	64.6	83.1	100.0	188.9	236.5
" " 28 " "	26.3	34.2	46.2	52.9	100.0	125.2
" " 3 Mo. "	21.0	28.3	36.9	42.3	80.0	100.0

NOTE.—This is an independent series and the strength ratios do not exactly agree with those in some other tests.

TABLE 11.—SERIES C1 (a), (b) AND C2 (a), (b); (STORAGE I FOR ALL); CONCRETE 28-DAY STRENGTH.

Series	$\frac{W}{C}$	Description	Age (da.)	Compr. Str. (lb. per sq. in.) 1	2	3	Av.	Variation from mean per cent	Relative Str. per cent of Std.
a	b	c	d	e	f	g	h	i	j
C1 (a)	1.10	In at.........	1 (std.)	1605	1513	1395	1504	4.9	100.0
			2	1182	1375	1175	1244	7.0	83.2
			5	1125	1153	1362	1213	4.0	81.3
			10	1090	1250	1176	1172	4.7	78.4
			20	1315	1061	976	1117	11.8	74.6
			25	854	1115	790	920	14.2	61.6
			27	781	779	878	813	5.2	54.5
			28 (dry)	814	1215	882	970	16.8	68.6
				Average Variation................				8.6	
C2 (a)	0.80	In at.........	1 (std.)	3560	3550	3390	3500	2.1	100.0
			2	3240	2840[1]	3038	3139	3.3	95.0
			5	3050	3059	3005	3038	0.8	92.0
			10	2710	2590	2730	2677	2.2	81.1
			20	2375	1730[1]	2540	2457	3.3	74.3
			25	2155	2300	2300	2252	2.8	68.1
			27	2005	2150	2360	2172	5.8	65.7
			28 (dry)	2670	2660	2580	2637	1.5	76.5
				Average Variation................				2.72	
C1 (b)	1.10	Out at........	28 (std.)	1705	1295	1448	1483	10.0	100.0
			27	1220	1518	1358	1365	7.4	91.4
			25	1480	1502	1413	1465	2.4	98.0
			20	1945	1350	1440	1578	15.5	105.7
			10	1670	1590	1505	1588	3.5	106.3
			5	1280	1465	1389	1378	4.8	92.2
			2	1130	1100	1268	1166	5.7	78.0
			1 (dry)	1040	1090	1109	1080	2.5	68.6
				Average Variation..................				6.5	
C2 (b)	0.80	Out at........	28 (std.)	3260	3330	2728	3106	8.0	100.0
			27	2920	3400	3233	3184	5.4	96.5
			25	3570	3660	3238	3489	4.9	105.5
			20	3970	3680	3460	3703	4.8	112.1
			10	3800	3840	3603	3748	2.5	113.4
			5	3100	3410	3278	3263	3.3	98.8
			2	3000	2385	2619	2668	8.3	80.8
			1 (dry)	2570	2310	2400	2427	4.0	76.5
				Average Variation................				5.1	

[1] Defective specimen. Omitted from average

The specimens of Table 10, although of the same materials and proportions as Series M 1 are extracted from a different lot of tests and Table 10 is included here only because it gives the detailed rate of strength

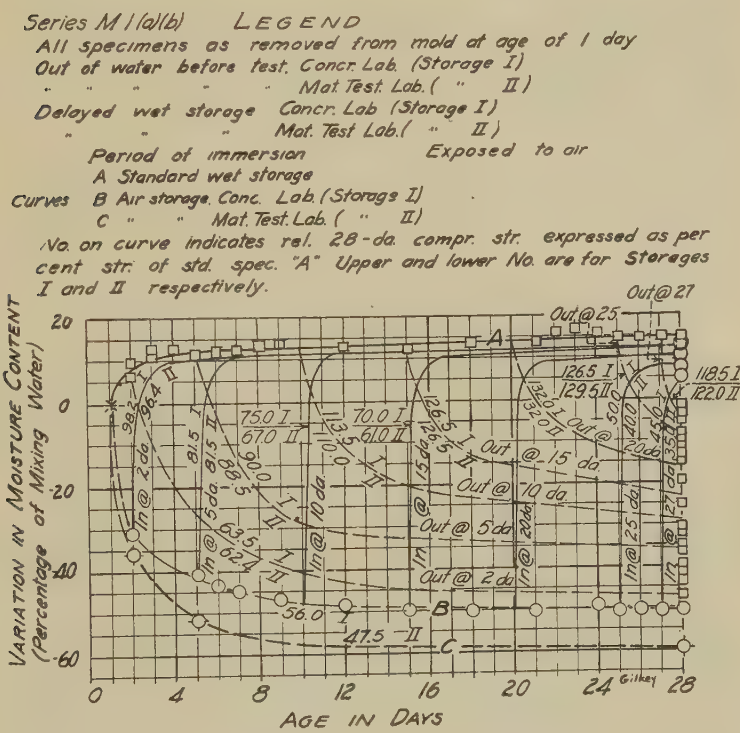


FIG. 1.—DELAYED STORAGE AND OUT BEFORE TEST: VARIATION IN WATER CONTENT FOR 28-DAY MORTAR SPECIMENS.

Each point represents the mean of 4 or more 2 x 4-in. mortar specimens. Proportions 1 : 2.5 by wt. $\frac{W}{C} = 0.70$. The only points shown are those corresponding to weight as removed from mold (1 day); as stored in, or removed from water and as tested (28 day). All the inner lines are drawn by interpolation and are for Storage I only. The lines for Storage II would be exactly similar but curve "C" would form the lower boundary.

increase with age for a mixture similar to the one used. Curve B (Fig. 13) is based mainly upon the data of Table 10.

Table 11 gives the data on the main concrete tests, Series C1 (a) (b) and C2 (a) (b). The two series were identical except for the water-cement ratio. These data were quite consistent, especially those for a $\frac{W}{C}$ of 0.8.

The results are all shown on curves and further treatment will be accorded in that connection.

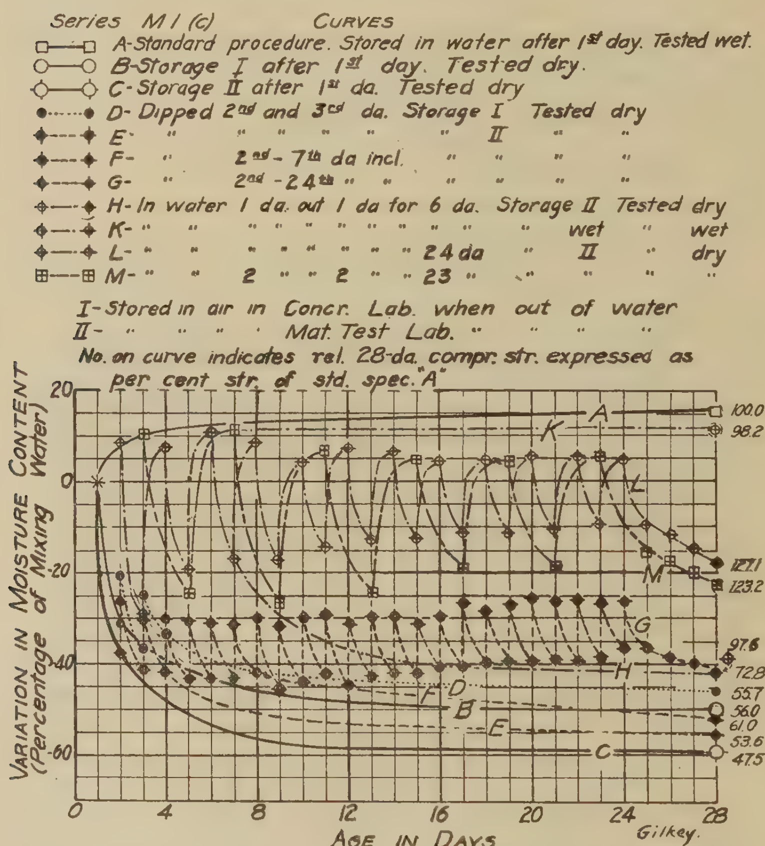


FIG. 2.—INTERMITTENT WETTING (MORTAR): VARIATION IN WATER CONTENT FOR 28-DAY SPECIMENS.

Each point represents the mean of four 2 x 4-in. mortar cylinders. Proportions—1 : 2.5 by weight; water-cement ratio = 0.70. *Note:*—Elsewhere in this paper there are indications that max. strength is attained after several days of drying out. Spec. tested wet (A and K) would show a considerable increase in strength if dry at test (for these proportions, as much as 30 to 40 per cent). Dry tested specimens would show a corresponding decrease in strength if soaked prior to test. This point should not be overlooked in making comparisons of strengths as given by dry-tested and wet-tested specimens.

Figures and Curves.—Figs. 1 to 5 inclusive give the detailed history of the soaking and evaporation processes for concretes and mortars of different proportions, water-cement ratios, and size of specimen. The effects of the

two kinds of air storage are shown. The moisture change is in all cases expressed as a percentage of the original mixing water which seems to be the fairest method for comparing mortars and concretes as well as variable water-cement ratios.

All specimens in Series M1 (a) and (b), Fig. 1, were weighed as removed from mold, which weight was used as basic in all cases (since the unknown weight of cap added, after making, invalidates the "weight as

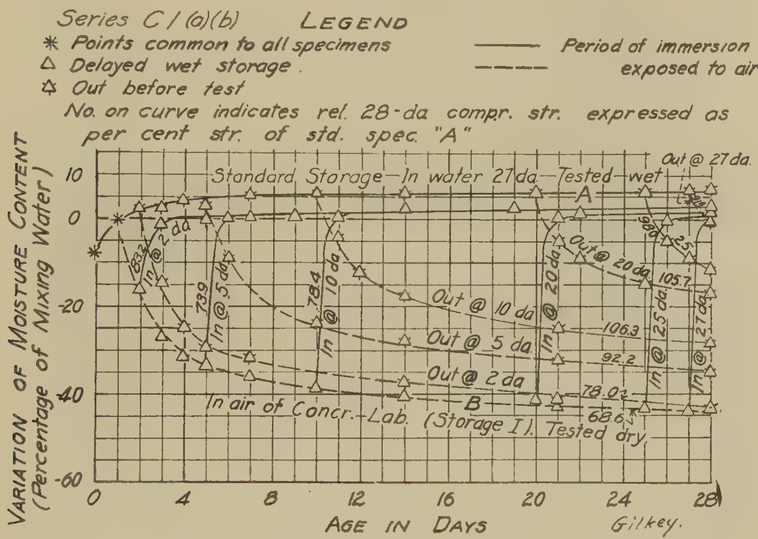


FIG. 3.—DELAYED STORAGE AND OUT BEFORE TEST: VARIATION IN WATER CONTENT FOR 28-DAY CONCRETE SPECIMENS.

$$\frac{W}{C} = 1.1$$

Each point represents from three to thirty 6 x 12-in. cylinders. Proportions 1:2:4 by loose volume. Proportioning was actually done by very accurate weighing, but the weights were selected to give an equivalent of approximately 1:2:4 by loose volume. The point at the left represents the weight as made. The next point represents the weight as removed from mold. The two weights are not comparable due to indeterminate weight of cap, leakage and evaporation. The weight as removed from mold (age of 1 day) is used as the standard for comparison in all cases. Subsequent weight changes are due entirely to decrease or increase in water content. When not immersed the specimens were stored in the air of the concrete laboratory (Storage I).

made" for comparison with subsequent weights). They were again weighed upon storage or removal therefrom, as the case might be, and once more at the time of test. These weights gave the experimental data surrounding the figure. The interior was contoured, for storage I only, by using the outside controls and by analogy with Fig. 2, 3 and 4, for which interior points were found by actual weighing. The 28-day strength ratios on the curves are of interest.

It should be noted that the specimens stored in the drier air (Storage II) lost more moisture than those stored in the concrete laboratory (Storage I) and that the strength ratio is lower. As much as 60 per cent of the original mixing water was lost, which means well under 40 per cent remaining since the specimen at one day does not have as high a water

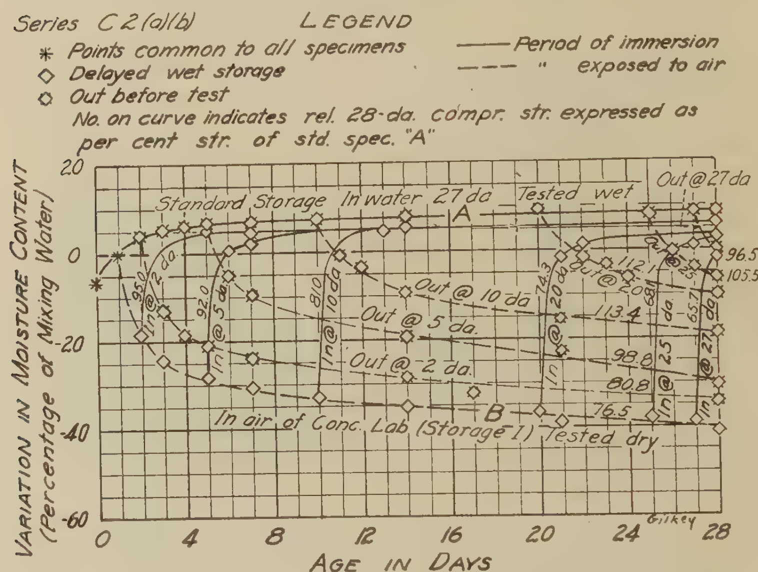


FIG. 4.—DELAYED STORAGE AND OUT BEFORE TEST: VARIATION IN WATER CONTENT FOR 28-DAY CONCRETE SPECIMENS.

$$\frac{W}{C} = 0.8$$

Each point represents from three to thirty 6 x 12-in. cylinders. Proportions 1 : 2 : 4 by loose volume (Proportioning was actually done by very accurate weighing, but the weights were selected to give an equivalent of approximately 1 : 2 : 4 by loose volume). The point at the left represents the weight as made. The next point represents the weight as removed from mold. The two are not comparable due to indeterminate weight of cap, leakage and evaporation. The weight as removed from mold (age 1 day) is used as the standard for comparison in all cases. Subsequent weight changes are due entirely to change in water content. When not immersed the specimens were stored in the air of the concrete laboratory (Storage I).

content as when mixed. Specimens that have partially dried out do not appear to quite regain the same degree of saturation as those never dried. Doubtless this is due to the presence of a certain amount of entrapped air that was not present before the first drying. This is shown by the upper boundary line of the "In" curves lying just below the saturation or "Standard Storage" line. Figs. 1, 3, 4 and 5 have a characteristic similarity in

shape but fundamental differences in amount of moisture change. The rate of moisture loss is rather gradual and at a diminishing rate. The steepness of the moisture loss curves (descending dashed lines) varies with the dryness of the air, and is steeper for small specimens than for large ones, as is to be expected. The moisture gain curves (ascending solid lines) are very steep for all mixtures until the moisture content approximates that of the specimen at the age of one day (removal from molds, in general). The additional rate of gain in moisture is then much slower.

In every instance, specimens, large or small, caught up with their weight as removed from mold in the first 24 hours of immersion. The early rapid gain in moisture is similar to the case with many stones, brick or tile.

Fig. 2 has upper and lower limits (envelopes) identical with Fig. 1. The effects of dipping and soaking for one day or two days are clearly shown. The dipping process was of only about 10 seconds duration, yet the 2 x 4-in. specimens took on almost instantaneously about 15 per cent of total original moisture content. The alternate soaking and drying curves show about the same upper boundary (water absorbed in two days' time to be about equal to one day absorption) but the two-day drying was always about half as great again as the one-day drying. All specimens with the exception of those of Curves K and A were allowed to dry out a few days before test. It is apparent that they did not all attain the same degree of dryness. As the number of immersions and removals increased, the range of moisture change for Curves L and M (Fig. 2) seemed to steadily diminish. Why this was so is not known or surmised.

The percentage strengths appearing at the right of the curves of Fig. 2 lead to the conclusion that dipping and soaking have little effect except in so far as they may be more or less equivalent to wet storage with interruptions. Gain of strength practically ceases when the mortar or concrete drys out. If the concrete is "in one day" and "out one," probably enough moisture is held for the day "out" to permit the curing to continue with practically no interruption. "Out two days," small specimens, or small bodies of concrete in dry air would have their curing halted, probably, for part of the time. Even the dipping gave a substantial increase in strength over the dry curing. It must be remembered that direct comparison between concrete tested wet and that tested dry cannot be made, with fairness.

The mere mechanical process of drying out, as previously stated, increases the strength from 10 to 35 per cent. This is not a permanent gain, however. The same concrete again wet would have only the strength of wet tested concrete. Thus a specimen wet at test is always potentially as strong as one just like it tested dry and giving a much higher test strength, while one tested dry is potentially as weak as the wet tested one. Thus we have an apparent contradiction, viz.: that the ideal curing condition is that of moist storage for the longest possible time, while the greatest strength is attained by removal from storage and drying out. Doubtless it is a fair analogy to liken saturated concrete to saturated stone or brick

either of which will test much weaker while wet than dry. (The analogy must end here since neither the stone nor the brick sets or cures.)

Thus true curing produces permanent improvement in strength, while mere drying produces a temporary increase, present only as long as the dry condition exists. This substantial increase in strength attainable by

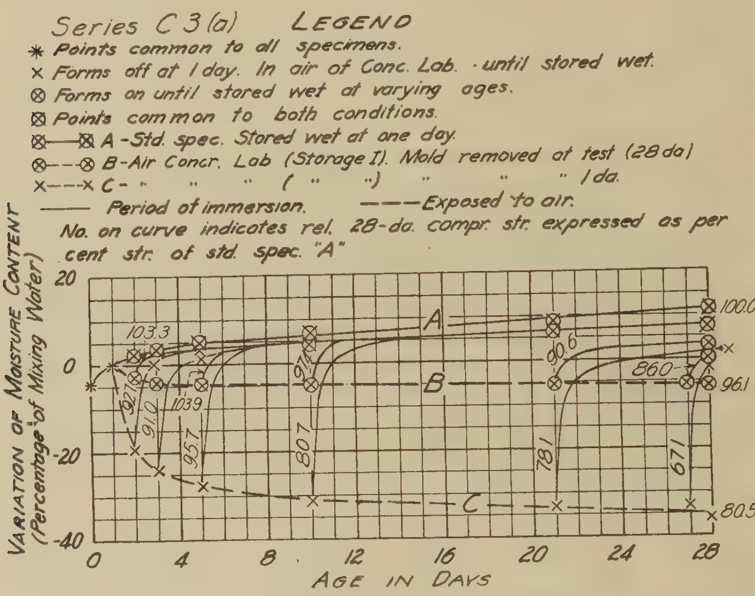


FIG. 5.—DELAYED STORAGE: FORMS LEFT ON UNTIL IMMERSION VS. FORMS OFF. VARIATION IN WATER CONTENT FOR 28-DAY CONCRETE SPECIMENS.

Size 6 x 12 in. Proportions 1:2:3 by absolute volume $\frac{W}{C} = 0.647$.

The point at the left represents the net weight as made. The next point represents the net weight at 1 day (Used as std. wt.). The two weights are not comparable because of indeterminate weight of cap etc. added after making, and prior to 1 day of age.

drying prior to test has been recognized and demonstrated by several.[1] It is not certain that it has always been clear in the minds of all that the gain is a purely mechanical one and that a dried out concrete removed from wet storage at an age of say 15 or 20 days and giving an ultimate strength 20 per cent. above that of a standard 28-day companion specimen, tested wet, is potentially about 15 per cent weaker than the 28-day specimen. The 28-day standard specimen has had the benefit of a week or more

[1] *Proceedings*, Am. Soc. C. E., May, 1925, p. 776, Conclusion No. 1 (W. K. Hatt). *Proceedings*, A. S. T. M., 1919, Part II, p. 608 (Howard W. Green). Bulletin No. 11, Lewis Institute, Fig. 7, or *Proceedings* A. C. I, 1922 (D. A. Abrams). Bulletin No. 2, Lewis Institute, Table 4, Fig. 5 (D. A. Abrams).

additional wet curing and if allowed to dry out before test would develop a strength of some 135 per cent of its own wet strength instead of the 120 per cent given by the specimen removed at 15 or 20 days. These figures will of course vary with the mixture the dryness of the air, mass or size of specimen, etc., but the general situation holds true.

In using or weighing the data of any of the figures or tables, if a comparison is being made between wet tested and dry tested specimens, the

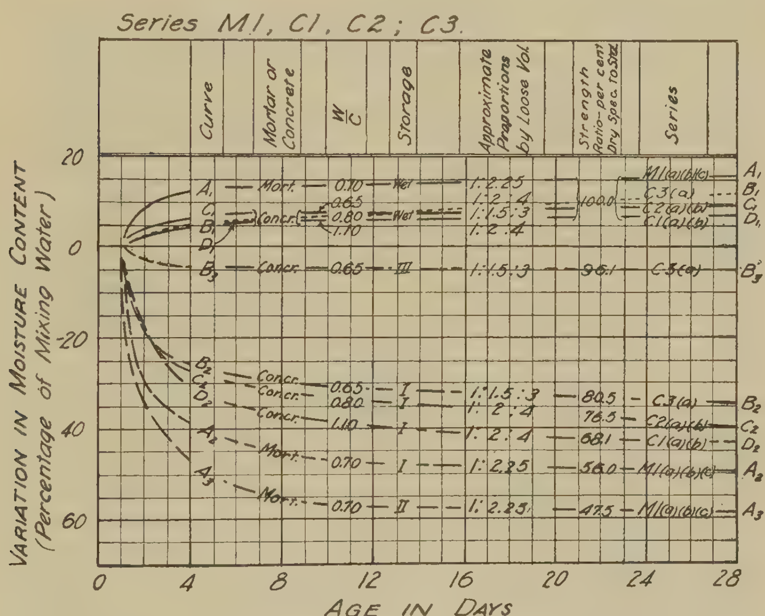


FIG. 6.—VARIATION IN WATER CONTENT: SUMMARY OF UPPER AND LOWER LIMITS FOR ALL 28-DAY CONCRETE AND MORTAR SPECIMENS.

Solid and dashed lines (except B) are periods in water and air respectively. These curves are the limiting cases of storage, viz., "wet" and "dry." Storages: I = Air in Concr. Lab., II = Air in Mat. Test. Lab. (much drier than that of Concr. Lab.), III = Form on until stored in water, Wet = std. storage. Series M 2 (a) not included, as no weights were taken. Loose volume proportions are closely approximate, being the reductions from proportions by weight or absolute volumes used.

strength shown for the dry tested specimen should be substantially reduced. Thus in Fig. 2, if the specimens of Curve C were immersed in water a day before test, they would have a strength ratio of 40, or so, instead of 47.5. Curves K or A if altered to compare on a dry basis would show ratios of about 135 per cent, the showing being quite favorable as compared with the next best case of "in a day" and "out a day" for 24 days.

Fig. 5 covers only the phase of "Delayed Storage." Curve B, for 6 x 12-in. specimens with the molds left on in the air of the concrete lab-

oratory (Storage I) show a very small loss of moisture and a correspondingly slight reduction in strength. This evidence along with that of Series M2 as shown by Curves A_3 and A_4 and by F (this series), all on Fig. 11, is quite conclusive that there is present in the concrete as mixed all the water that is needed for curing purposes. If this water can be conserved, the concrete will increase in strength in the same manner and

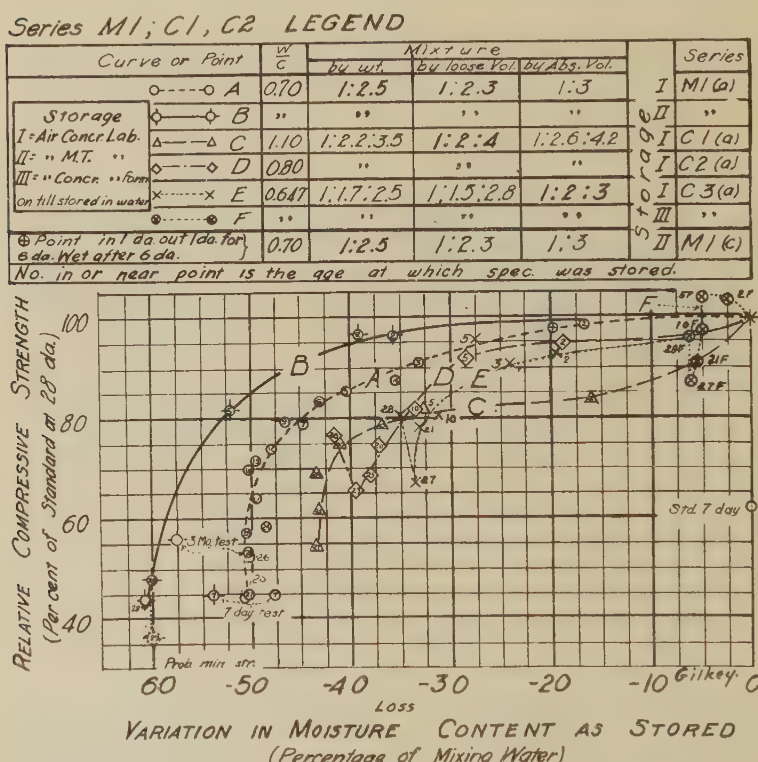


FIG. 7.—DELAYED STORAGE: MOISTURE CONTENT AS STORED VS. COMPRESSIVE STRENGTH.

Each point represents from 1 to 6 specimens.

degree as it would if stored moist or wet. There is already general agreement that moist sand, or burlap, damp room, and water storage are identical in result. Any one of them furnishes all the water needed. This added information would make it appear that even leaving on the forms would in many cases vastly improve the quality of the concrete. It is not unreasonable to suggest that a surface paint or coating of some sort might be devised to seal the surface of the concrete and hold in the water, thus

making the initial moisture effective for curing. The use of calcium chloride and similar agents to attract moisture from the air for curing purposes is of course along this general line, but from a somewhat different angle.

Fig. 6 summarizes the results of the preceding figures (except 2). From it the following conclusions may be drawn:

1. Small specimens or thin bodies of concrete will fluctuate in moisture content and strength ratio much more than large specimens, or masses of concrete.
2. The drier the air the greater the loss of moisture and lowering of strength.
3. Mortars will probably fluctuate more than concretes (not decisive from the data at hand).
4. Wet mixed concretes have a greater fluctuation in water content than dry mixed concrete.
5. Wet mixed concretes are more vulnerable to lack of moisture in curing than dry mixed concretes.
6. Keeping the outside surfaces of concrete sealed from the air will effectively prevent moisture losses and produce nearly as favorable a curing condition as actual moist storage.

Fig. 7 shows compressive strength plotted against loss in moisture content (expressed as percentage of original mixing water) at the time that "delayed storage" specimens were immersed. For any one condition of storage or any one mixture of concrete or mortar, there seems to be a uniform relation between strength and moisture content. But the relation is not constant for different mixes and different storages. The curves, though interesting and worthy of rather careful study, do not appear to define any general law or relation. It may be that this information is amenable to interpretation and correlation with other data in ways not now apparent.

Fig. 8 is analogous to Fig. 7 but shows strength plotted against moisture change (from the standard one-day weight, expressed as the usual percentage of original mixing water) at the time of test for the "out before test" specimens.

The showing here is more positive than that of Fig. 7. For all but the very wet concrete ($\frac{W}{C} = 1.1$, Curve C) as a straight line function until 15 per cent of the original mixing water has been lost. Then the strength starts down hill following (in the case of the mortars) a different straight line course for each of the two storage conditions.

The concretes act in a manner quite similar to the mortars but the change in strength for a given change in moisture is not as great. One interesting point in the curves for the concrete is an evident falling off in strength for the lower moisture losses, then a steady gain until the maxima

are reached at 15 per cent moisture loss for $\frac{W}{C}$ of 0.80 and 23 per cent loss for a $\frac{W}{C}$ of 1.10. A similar tendency was indicated in a very slight degree for mortars removed only a few hours before test. Just why the

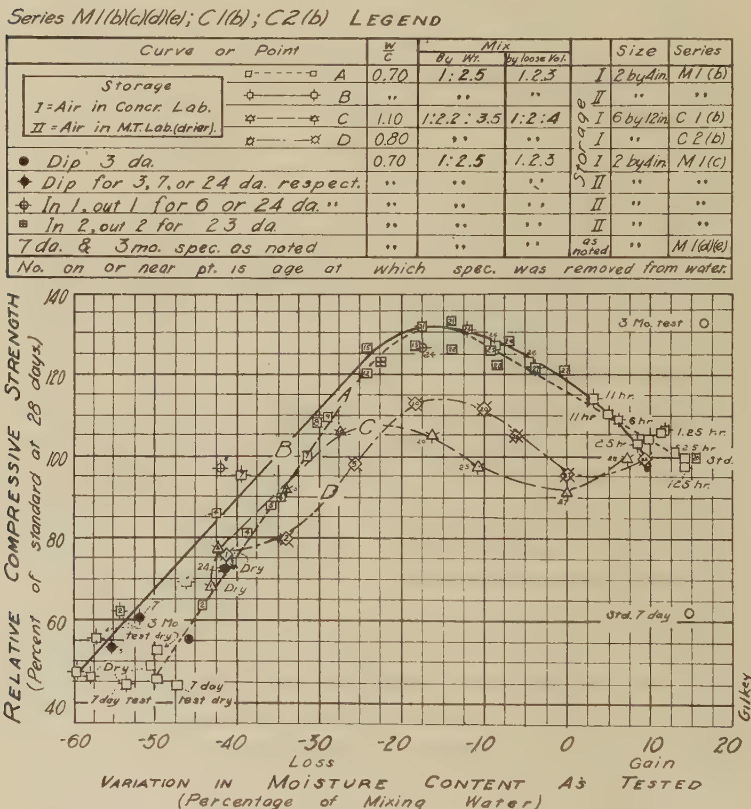


FIG. 8.—OUT BEFORE TEST: MOISTURE CONTENT AT TEST VS. COMPRESSIVE STRENGTH.

Each point represents from 1 to 6 spec.

strength should fall off in the early stages of drying out is not apparent unless it be due to unequal hardening about the exterior of the specimens (because of unequal rates of evaporation). This unequal hardening might cause lack of homogeneity sufficient to give the equivalent of eccentric loading. When more complete drying out had occurred the strength of the

material would again be uniform and the equivalent of eccentric loading conditions would no longer exist. From this reasoning, the small specimens would show this in a much less degree because of the rapidity with which they dry. These same phenomena have been noted by others but the reasons therefor seem still to be a matter of conjecture. Thus, Fig. 8 resembles Fig. 7 in showing a harmony of action for any one mixture and condition of storage without apparently defining any general law.

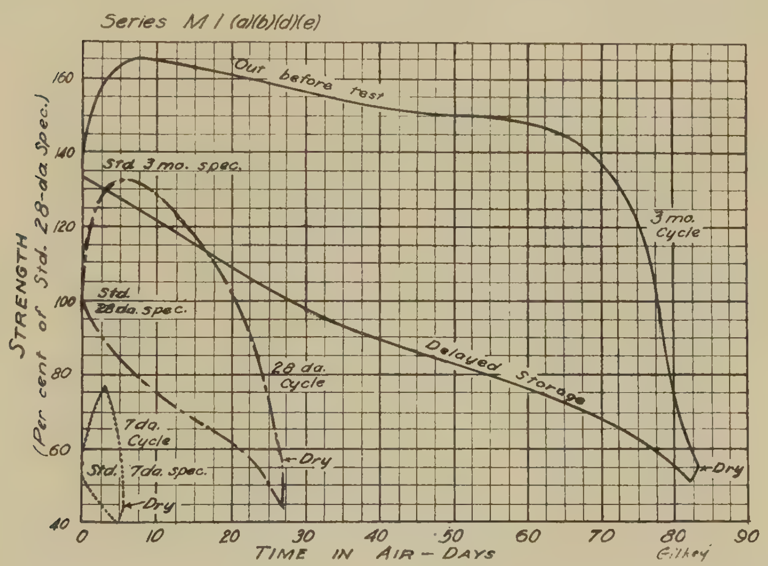


FIG. 9.—STORAGE VS. STRENGTH (CYCLES): DELAYED AND INCOMPLETE WET STORAGE FOR 7-DAY, 28-DAY, AND 3-MO. PERIODS ALL REFERRED TO STD. 28-DAY SPEC.

Notes: These are generalized curves for the mortars of series M 1 (a) (b) (d) (e). The 7-day and 28-day cycles are completely covered by experimental data. The 3-mo. curve is not fully surrounded, but is approximately correct.

Figs. 9 and 10 are very interesting portrayals of the manner and extent to which exposure to air both before and after wet storage effect strength. In studying these two curves it should be kept in mind that the horizontal scale shows days exposed to air *after wet storage* and just *prior to test* for the upper curves while for the lower curves it shows days exposed to air *after removal from molds* and *prior to wet storage.* Upper curve specimens were dry at test except the limiting case at the left "removed from water at zero days before test," (in other words "never removed until test").

Lower curve specimens were all tested wet except the limiting case at

the right "stored in water at 6 days, 27 days or 3 months less one day" (in other words "never stored"). The ages at which upper curve specimens were removed from wet storage are therefore the age at test less the horizontal scale reading. The age at which lower curve specimens were stored is the horizontal scale reading plus one day (the specimen was in the mold for one day).

The discussion in connection with Figs. 1 to 5 inclusive applies full force to Figs. 9, 10, 11 and 12. The strength loss due to delay in storage is very real in all cases. It is a loss that possibly cannot be fully recovered, (as is shown by Fig. 13, to be discussed later). On the other hand, the ap-

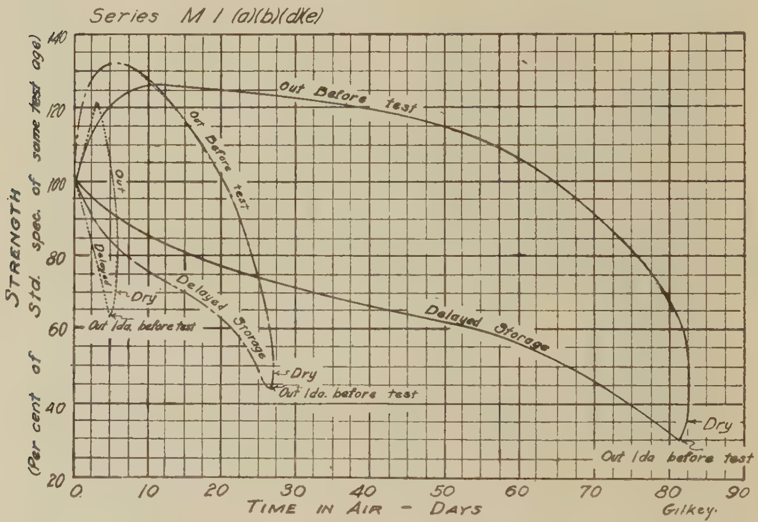


FIG. 10.—STORAGE VS. STRENGTH (CYCLES): DELAYED AND INCOMPLETE WET STORAGE FOR 7-DAY, 28-DAY, AND 3-MO. PERIODS REFERRED TO STD. SPEC. OF SAME TEST AGE.

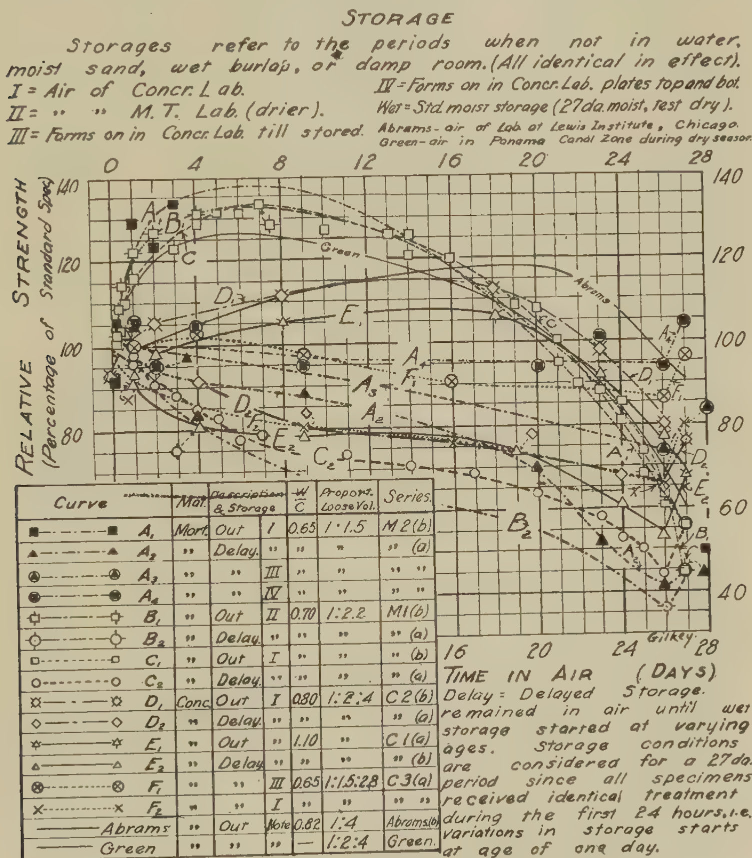
Notes: These are generalized curves for the mortars of series M 1 (a) (b) (d) (e). The 7-day and 28-day cycles are completely covered by experimental data. The 3-mo. curve is not fully surrounded, but is approximately correct.

parent substantial gain in strength by premature removal from moist storage is not real in the sense explained several pages back.

The only difference between Figs. 9 and 10 is that strengths in Fig. 9 are expressed in terms of the strength of a 28-day standard specimen and in Fig. 10 they are expressed in terms of the strength of a well-cured wet tested or standard specimen of the same test age.

Any specimen along the upper curves would show a substantial reduction in strength if soaked just prior to test. None would have a strength as high as the specimen stored in standard manner. (100 per cent in Fig.

10 or 133.3, 100, or 62.4 per cent, respectively, in Fig. 9). Moreover any specimen along the lower curve would show a considerable increase in strength if dried out before test. The strength of the standard specimen



Curve	Mat.	Description & Storage		W/C	Proport. Loose Vol.	Series.
A_1	Mort.	Out	I	0.65	1:1.5	M 2(b)
A_2	"	Delay.	"	"	"	" (a)
A_3	"	"	III	"	"	" "
A_4	"	"	IV	"	"	" "
B_1	"	Out	II	0.70	1:2.2	M1(b)
B_2	"	Delay.	"	"	"	" (a)
C_1	"	Out	I	"	"	" (b)
C_2	"	Delay.	"	"	"	" (a)
D_1	Conc.	Out	I	0.80	1:2:4	C 2(b)
D_2	"	Delay.	"	"	"	" (a)
E_1	"	Out	"	1.10	"	C 1(a)
E_2	"	Delay.	"	"	"	" (b)
F_1	"	"	III	0.65	1:1.5:2.8	C 3(a)
F_2	"	"	I	"	"	" "
Abrams	"	Out	Note	0.82	1:4	Abrams(b)
Green	"	"	"	—	1:2:4	Green.

FIG. 11.—STORAGE VS. 28-DAY STRENGTH: EFFECTS OF DELAYED AND INCOMPLETE WET STORAGE.

Loose volume proportions are in round numbers. For exact proportions see Table 1. Out = Stored wet at 1 day, removed from water at varying ages prior to test.

would in each case reach the top and slightly surmount the highest point in the upper loop.

The specimens "stored one day prior to test" are invariably weaker than the specimens never stored, due to the weakness while in a soaked condition. The curves illustrate this, the minimum strength always occurring on the lower curve one day short of testing age.

Fig. 11 is a somewhat congested but very interesting group of loops (or half loops in some cases in which data for only a half cycle were taken). These are all 28-day cycles, exactly like those shown in Figs. 9 and 10. All the series of these tests [except M3(a) for which was taken no such data] are shown. Moreover, the tests of both Abrams and Green are plotted here for comparison. The latter tests are both of the "out before test" phase which seems to have been the only phase considered heretofore. The agreement of the curves in general trend is remarkable when the range of locality, mixture, etc., are considered. All the delayed storage curves show the weakness of the specimen placed in storage the day before test. Curves A_3 A_4 and F_1 show the great gain in strength attainable by leaving the forms on specimens exposed to the air, (delayed storage) thus restricting if not entirely stopping moisture loss by evaporation.

Mortars apparently respond in a greater degree than concretes (but this may be due to difference in size of mortar and concrete specimens, with a more complete drying of the former).

The 2 x 4-in. mortar specimens (out before test, i. e., upper series of curves) reach their ultimate strength at fewer days of exposure than do the 6 x 12-in. concrete specimens. This is easily explainable again by the differences in rates of drying out for different sized specimens. Moreover, all concrete specimens of these series were stored in the air of the concrete laboratory (Storage I) which was not as dry as that of the materials testing laboratory (Storage II) in which many of the mortar specimens were stored.

The peak strength for Green's tests is reached at an exposure of 5 or 6 days (out at about 21 or 22 days). The maximum strengths for the concretes of our tests (Series C 1(b) and C 2(b)) are both reached at about a 16-day exposure (out at about 12 days of age) while Abram's peak comes at 20 days' exposure (out at about 8 days of age).

These differences appear to be easy of explanation. Green's tests were conducted in the Panama Canal Zone during an extremely dry season of the year. Rapid drying out of concrete naturally resulted. The air in Chicago and especially in the laboratories is generally quite humid, as compared with either the Panama Canal Zone, or Colorado, and the drying rate correspondingly slower.

The decrease in strength because of delayed moist storage has an approximately straight line variation between the 100 per cent for the standard specimen stored wet at one day and tested wet to 50 per cent for the dry specimen tested dry. Both Figs. 11 and 13 show that a specimen exposed to the air until the fourteenth day will develop about 85 to 90 per cent as much strength as the standard specimen if tested in the same condition. On the other hand, if the mold be left on till the fourteenth day it will develop from 85 to 96 per cent of the standard strength.

The upper loop of A_1 and lower loop B_2 were not fully determined and the maximum and minimum values may not be exactly as shown. Both ends are controlled, however.

Fig. 12 is a set of 4-mo. tests by Abrams and 3-mo. tests from this series.

The "out before test" curves (A, B, C, D, E, and F) are reversed in direction from those of the preceding figures. The horizontal scale represents the period in moist storage before exposure to the air and not the period in air following moist storage. These curves show the effect of a

Series M 1(e)(f); Abrams(a) LEGEND

Curve	Age at test Months	Description	Mortar or Concrete	Proportions Loose Vol.	W/C	Series
A	3 mo.	Out	Mort.	1 : 2.25	0.70	M 1(e)(f)
B	4 "	"	Concr.	1 : 4	0.66	Abrams(a)
C	4 "	"	"	"	0.81	" "
D	4 "	"	"	"	0.99	" "
E	4 "	"	"	"	1.09	" "
F	3 "	" (Aut.)	Mort.	1 : 2.25	0.70	M1(f)
G	3 "	Delay "	"	"	"	" "
H	3 "	"	"	"	"	" (e)(f)

Out = early water storage (dry at test).
Delay = " air " (wet " ").
Abrams = Bull. 2. Lewis Inst. Table 4 p. 7.

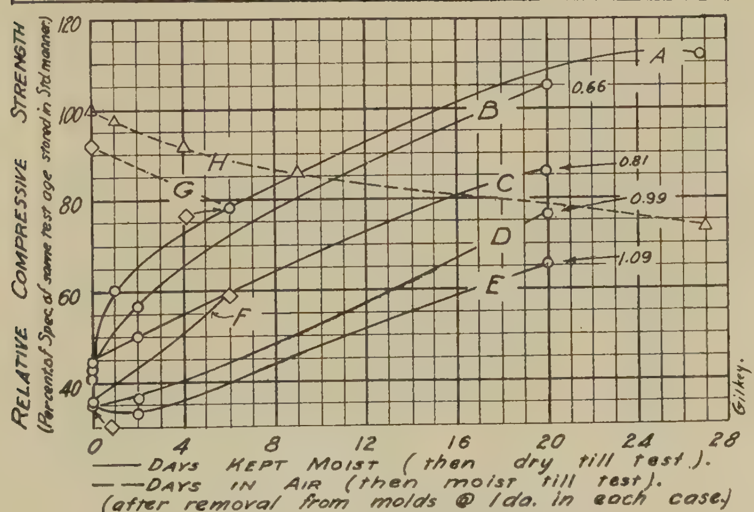


FIG. 12.—STORAGE VS. STRENGTH: EFFECT OF EARLY WETTING AND EARLY DRYING OUT ON 3- AND 4-MO. STRENGTH.

short time in water and a long time in air. It is evident that both the 3- and 4-mo. strengths are almost directly proportional to the period of moist curing, between the limiting strengths of the dry-cured specimen on the left and the specimen cured moist for three or four weeks (but tested at three or four months) at the right.

The dashed curves, G and H, are delayed-storage strength graphs and show the effect of delayed storage, (for periods of delay up to 4 weeks)

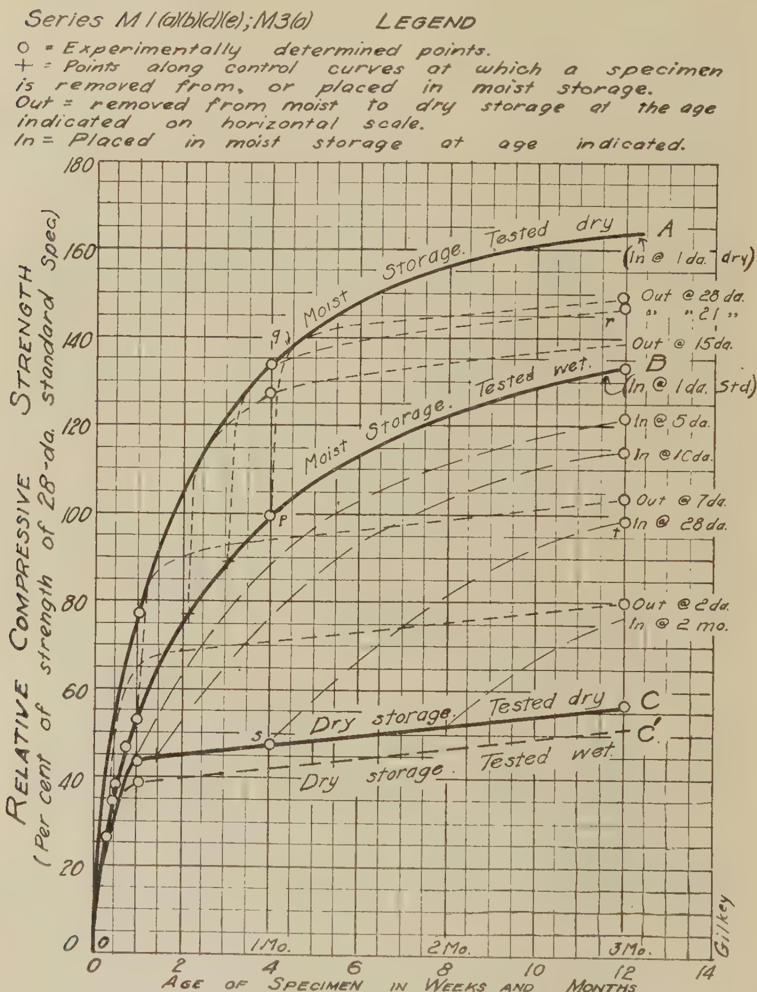


FIG. 13.—UNIVERSAL CURING DIAGRAM: ILLUSTRATIVE GRAPH FOR PREDICTING RELATIVE COMPRESSIVE STRENGTHS OF MORTARS AND CONCRETES AT ANY AGE AND FOR ANY COMBINATION OF WET AND DRY STORAGE.

The vertical scale will vary for different mixtures, etc. A single graph could be made to cover a wide range of usual mixtures with scant error. The time scale could be extended by further experiment as far as desired. The present graph is based on series M 1 (a, b, d, e). [Mortars 1:2.5 by wt. (or 1:2.26 by loose vol.) $\frac{W}{C} = 0.70$]. As shown by Fig. 11, the percentage strength change is greater for mortars than for concretes and for small spec. than for large ones. These spec. were 2 x 4 in.

upon the 3-mo. strength. They are the same as the lower boundary line of the 3-mo. cycle of Fig. 10. The evidence from Fig. 12 fully accords with that from the other data of the tests.

In investigational work there is always present consciously or otherwise, the hope that the experiments may lead to some general law or simple key to the whole matter under observation. It is only an occasional project that ends that way. It appears as if this were one of these favored efforts. Fig. 13 was evolved from these data in a very natural manner. It seems adequately to cover the whole problem of water and the curing process. This figure is therefore offered for your use and criticism. It is not expected nor claimed that the percentages as determined by a mortar series in Colorado climate will correctly fit your particular concrete. But you can make a similar chart from your own concrete that should serve as a universal curing diagram that will give reliable strength data for any probable "water and air" curing condition. The diagram is fully explained and its use illustrated in the notes that accompany it. The strength scale may be in terms of any standard strength or percentage. For example, if it were desired to check the data of Fig. 12 by the diagram it would be necessary to change the ordinates of Fig. 12 to percentage of 28-day strength instead of percentage of strength of the specimen 3 (or 4) months old stored in standard manner (in water entire time after remaining in mold for one day). Either Fig. 12 or Fig. 13 could be altered to suit the other.

In the early parts of the curve a slight difference in rate of drying will have a relatively large influence upon the location of the point of tangency between the drying curve and the upper envelope because the two curves are nearly parallel. Further on the probable error is not great.

In connection with Fig. 13 it should be emphasized once more that the

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USE OF GRAPH.

I. What probable percentage of 28-day standard strength will be developed by this mortar in 2 weeks?
   (a) If stored moist and tested wet?
   (b) If stored moist and tested dry?
   (c) If stored dry and tested dry?
   (d) If stored dry and tested wet?

II. What will be the percentage for 1, 2, 3-mo. strengths?
   (a) If removed from water at 28 day?
      (1) Tested dry? (2) Tested wet?
   (b) If not placed in water until 28-da. old?
      (1) Tested dry? (2) Tested wet?

RESULTS

I. Reading values directly from curves *B, A, C,* & C^1 at two weeks:
   (a) 75.; (b) 112.; (c) 45.; (d) 40.

II. (a) *opgr* represents percentage strengths.
      (1) 1 mo. test dry 134; 2 mo. 145; 3 mo. 149.
      (2) 1 mo. test wet 100; 2 mo. wet = 145 — pg, or more accurately,

$$145 - \frac{pg + AB}{2} = 145 - 34 \pm = 111 \pm; \quad 3 \text{ mo. wet} = 149 - 30$$

$= 119.$

   (b) *ost* represents percentage strengths.
      (1) 1 mo. dry 47; 2 mo. 81; 3 mo. 98.
      (2) 1 mo. wet 47 — CC^1 = 47 — 5 = 42; 2 mo. wet 81 — CC^1 = 76; 3 mo. wet 98 — 5 = 93.

large excess strength along curve OA over that along the standard curve OB is not generally a usable gain because the differential simply represents a drying out process that is reversible. Any specimen of any strength along OB may have its strength increased to that of the point nearly directly above on OA curve by a few days of exposure to dry air. But any strength along OA will be reduced to the one below it on OB by a very short period of soaking.

From the nature of the results and conclusions reached from these tests, as well as from much that has long been common knowledge, it is evident that any rule that will show a true relationship between 7-day and 28-day strengths for standard curing conditions will fall down for most curing conditions that are not standard.

In advancing a rule $f' = f + 30\sqrt{f}$ for predicting 28-day strength from 7-day tests, Professor Slater (in his paper before this convention) specifically excludes its application to cases in which curing conditions are not standard. The correctness of this stand is evidenced by the following tabulation from these tests:

		Strength		(Lb. per sq. in.)
Series	Curing Condition	Actual 7 day	28 day	28-day predicted $f' = f + 30\sqrt{f}$
M3 (a)	Standard	2414	4556	3900
M1 (a)	"	2620	4200	4170
"	In @ 2 da. I	2430	4130	3920
"	" " " II	2290	4040	3750
"	" 3 " I	2230	3805	3650
"	" 4 " I	2065	3665	3450
"	" 5 " I	1815	3575	3100
"	" 5 " II	1920	3420	3250
"	" 6 " I	1650	3500	2880
"	" 7 " I (dry)	1871	3260 (1)	3175
"	" 7 " I (dry)	1871	2350 (2)	3175
"	" 7 " II (dry)	1875	1996 (2)	3185
M1 (b)	Out @ 7 " I (7 day Std.)	2620	4210	4160
"	" " " II " "	2620	4015	4160
"	" 3 " II	3150	3610	4850
"	" 4 " I	3240	3115	4960
"	" 4 " II	2940	2896	4580
"	" 5 " II	3100	2620	4780
M1 (c)	In 1 out 1 for 6 day II	2760 (2)	4100 (2)	4340
"	Dip 2nd-7th incl. II	2220 (2)	2560 (2)	3650
"	Dip 2nd-3rd incl. II	2400 (2)	2253 (2)	3880

(1) Test Wet

(2) Test Dry

The "delayed storage" for periods of delay of 6 days or less agree very well with Slater's rule. As the delay becomes greater the agreement will be less perfect and the dry specimens give actual strengths at 28 days about 30 per cent below the predicted ones.

For the "out before test" series, the actual 28-day strength is at least 30 per cent below that predicted for most cases.

In the "intermittent wetting" series the "in one day out one day," etc., give excellent agreement, as the treatment is practically the equivalent of continuous wet storage on account of residual moisture held over. The dipped specimens on the other hand, are in about the same class as air-cured ones and they fall about as far short of developing the predicted 28-day strength.

Although it is evident that Professor Slater's rule will not apply to cases of variable storage, a diagram similar to that of Fig. 13 can be used to give the percentage relation between 7-day and 28-day strengths for any known condition of storage, and satisfactory 28-day (or any other period) predictions may be made if the 7-day strength (or any other convenient strength) be first determined.

At the beginning of this paper a list of questions was propounded. That list will be used as a basis for summarizing the conclusions from these tests.[1]

Q. 1. For how long should concrete be kept moist after being placed?

A. The longer the better, which you already know. To be more specific, we refer to Fig. 13, and find that dry after 7 days will give a 28-day strength, about 65 or 70 per cent that of the standard 28-day specimen in the same condition (wet or dry) at test. Dry after 14 days will develop about 88 per cent of the standard.

Q. 2. Is intermittent wetting or sprinkling as effective as genuine moist curing?

A. No, unless the wetting be thorough enough to keep the concrete moist during the interval between wettings. Whenever concrete dries, curing (i. e., gain in strength) almost ceases. It resumes at a diminished rate when moisture is again available.

Q. 3. Is an early drying out permanently detrimental to the concrete? (a) If the concrete be subsequently subjected to standard moist conditions? (b) If the concrete remain in the dried out state?

A. (a) Probably not. The concrete will resume the curing process

[1] It should be remembered that for such answers as are based on the universal curing diagram for Series M1 of these tests, we are using figures derived from mortar tests on small volumes (2 x 4-in.) exposed to Colorado air. Mortars are likely to have a higher percentage range of variation than concretes and the drying out of small specimens is relatively rapid. These answers must therefore in many instances be considered as relative, illustrative and tentative, rather than exact and final. In the case of thin slabs or joists in buildings heated against frost it is entirely possible that conditions even worse than those which Fig. 13 summarizes, may exist. On the other hand, the results from Fig. 13 will be rather too drastic for large beams, girders, etc., in this moist climate when artificial heat is not used. Pavements invariably cure better than is expected because of the moisture available from the sub-base. Construct your own diagram from the results of tests on your own concrete, and the figure will give you the correct numerical answers to your questions.

and may eventually "catch up." From Fig. 13 we see that a specimen that was in air after the first day then immersed at the age of one month had acquired a strength of about 98 per cent (of the 28-day standard) at the end of 3 months. This illustrates the diminished rate of gain. (b) Permanently and extremely detrimental. The concrete never will develop a strength appreciably above what it had at 5, 6, or 7 days (when enough of the original mixing water was evaporated to halt further curing action).[1]

Q. 4. Will alternate saturation and drying out give a concrete stronger or weaker than one subjected to one condition or the other?

A. Answered partially under 2. If the drying is not too prolonged the curing will be equivalent to a uniformly moist condition. If drying is complete between saturations strength will be reduced from that of moist cured concrete, but stronger than dry. (See series Ml (c) and Figs. 2 and 5.) This answer takes no account of some situations in which alternate swelling and shrinking might damage the concrete by too violent internal stress fluctuations, etc. Like the case of internal stresses due to temperature changes, this becomes a problem for the designer. This phase of the question might well be referred to Professor Hatt and others who are now conducting important researches along these lines.

Q. 5. Concrete cured moist is stronger than that dry cured, but is the difference enough to be of practical interest?

A. Yes. Concrete moist cured will develop from 1.5 to 3 times the 28-day strength of dry cured concrete, if both be in the same condition at test. The ratio becomes greater for greater ages.

Q. 6. Concrete cured moist but dried out at test is stronger than the same concrete tested wet, why, and to what extent?

A. Due to a mechanical hardening in the same way that dry brick or porous stone is stronger than the same material when saturated. When re-soaked the concrete resumes its former wet strength. The strength gain from drying out will vary from 10 to 40 per cent, dependant upon the mixture, degree of drying, etc. A good average value for typical concrete mixtures under average conditions may be taken, conservatively as 15 or 20 per cent. For wet mixtures (high $\frac{W}{C}$) it may be as low as 10 per cent.

Q. 7. Is moist storage as essential to wet mixed concrete as it is to drier mixes?

A. More so. Figs. 6 and 11 show that the air-cured specimens of concretes with $\frac{W}{C}$ of 0.65, 0.80, and 1.10 developed 80.5, 76.5, and 68.1 per cent, respectively, of the standard 28-day compressive strengths of the mixtures. These percentages would be 60.0, 57.5, and 51, respectively, if the standard specimen be in the same condition as the others (wet or dry) at test.

[1]These findings fully accord with those of H. F. Gonnerman, as reported in The A. C. I. *Proceedings*, Vol. XIV, 1918.

Q. 8. Do mixes of different richness respond in the same way and to the same extent, to different curing methods?

A. In the same way yes, but probably not to the same extent. These tests do not indicate clearly which way the trend lies. There is no series in which cement content, or richness of mix, was the only variable.

Q. 9. Is there any essential difference between the curing of concretes and mortars?

A. Not in manner, but some of these results appear to indicate quite a difference in degree. Mortars appear to have higher percentages fluctuation of strength, moisture content, etc., than do concretes. This evidence is complicated, however, by the presence of a pair of variables. The mortar and concrete specimens were not usually of the same size. (Mortars 2 x 4 in. and concretes 6 x 12 in.)

Q. 10. How is curing affected by size of specimen or body of concrete?

A. For a small specimen or small body of concrete with relatively large surface area, serious damage from drying out is to be expected. The response to an unfavorable environment is much more rapid and complete for small than for large specimens. The response to the addition of moisture is practically speaking, about equal in the two cases. Even a large mass of concrete soaks up moisture quite rapidly until nearly satisfied. (See Figs. 1 to 5 incl.)

Q. 11. If one of a series of specimens has been left exposed to the air a day or more longer than its mates (possibly for recapping, as sometimes occurs) will this specimen differ in strength due to the difference in treatment? (a) If the mold were left on during exposure? (b) If the mold were removed and the specimen in contact with the air during the exposure?

A. (a) Very slightly if at all. (b) Yes. The extent will vary with atmospheric conditions, and with size of specimens, but it will be appreciable in any case. The introduction of such variables should be avoided.

Q. 12. In building construction, would leaving forms on for longer periods assist in curing?

A. Probably. Too much reliance should not be placed upon it, however, as much evaporation can still occur from the upper surfaces. It is at least on the safe side.

Q. 13. Should special precautions be taken in building construction to avoid drying out, especially if heat be used to prevent freezing?

A. Yes. It is probably the rule that concrete in buildings (especially those erected in cold weather) is much weaker than the tests on moist cured samples indicate. (See *Proceedings*, Am. Soc. C. E., January, 1925, Slater and Walker, Table 19, Field Tests of Concrete), also (*Proceedings*, A. C. I., 1923, Discussion by P. J. Freeman, p. 39). It is very important that precautions be taken to supply plenty of moisture for curing.

Q. 14. Some laboratories habitually remove specimens from moist storage one day before test. Will this procedure permit a fair measure of relative strengths of (a) Specimens of different sizes (as the standard 6 x 12-in.; 8 x 16-in. and 2 x 4-in. cylinders). (b) Specimens tested at different seasons of the year or at different locations where evaporation rates vary?

A. (a) No. The extent of drying out is bound to differ. (b) No. Ditto.

Q. 15. Is the strength change proportional to the change in moisture content: (a) If the moisture be lost before moist storage starts? (b) If the moisture be lost after moist storage but prior to test? (c) If equal moisture losses in either of the above cases be attained at different rates, i. e., if a specimen attain a given loss (1) at a rapid rate, or (2) at a slow rate?

A. (a) (b) (c) Partially so. See Figs. 7 and 8 and the discussion of them.

Q. 16. How do rates of evaporation compare with rates of absorption (a) For specimens of the same size? (b) For specimens of different sizes?

A. (a) Rate of absorption always rapid until saturation point is nearly reached. Evaporation rate varies with the environment, but much slower than absorption in any case. (b) Difference in absorption rate not important, i. e., still relatively rapid for both, but rate of evaporation greater for smaller specimens.

Q. 17. How does the curing environment affect the phenomena of "autogenous healing"?

A. Autogenous healing will take place as long as moisture is present. It ceases whenever the concrete dries out, but will be resumed upon subsequent wetting.

Q. 18. To what extent do the results from these tests accord with other similar tests?

A. So far as known, similar tests have only been conducted to determine the effect of drying out after wet storage and prior to test ("out before test" series). In other words, one-half the cycle has had some former experimental attention. The works of Abrams and Green accord very perfectly with the results of these tests. Many of the conclusions from these tests are by no means new knowledge. On the other hand few attempts appear to have been made to systematically cover the general subject of curing in one treatment.

Q. 19. Will any rule for relation between 7-day and 28-day strengths hold true for variable curing conditions?

A. It will not since no fixed mathematical relation exists between 7-day and 28-day strength for different curing conditions. In case of dry air curing, for example, the 7-day and 28-day strengths are almost equal. Slater excludes his rule for use in any cases except those of

standard curing. A rule could doubtless be devised for any *one* condition of curing.

Q. 20. Can the subject of curing be condensed into a general law or basic principle that will enable the concrete practitioner to predict with reasonable accuracy, the strength of an otherwise known concrete, under a variety of known or assumed curing conditions?

A. Yes. Fig. 13 is illustrative of how this may be accomplished.

DISCUSSION.

Mr. McKesson. C. L. McKesson (*By Letter*).—It should be borne in mind in studying this carefully-prepared paper by Prof. Gilkey, that the major portion of the test specimens consisted of 2 x 4-in. mortar cylinders. Our experience in studies of the curing of concrete has not indicated that results obtained from curing tests on small mortar specimens are indicative of results to be had in curing tests of larger concrete specimens or of concrete in mass in the field. The small mortar specimens after removal from water or moist storage undoubtedly dry out much more rapidly than larger specimens or mass concrete and the water needed for hydration is, therefore, lost much sooner.

That 2 x 4-in. mortar specimens and 6 x 12-in. concrete specimens do not give parallel results in curing tests is indicated by Table 11 in Pro. Gilkey's paper in which concrete specimens, Series C1 (b) and C2 (b), show greatest 28-day strength with a wet curing period of 10 to 20 days. This is also shown in Fig. 11 in which curves D_1 and E_1 show this same result graphically. The Abrams curve in Fig. 11 shows a maximum 28-day strength for concrete when specimens are removed from water at about 8 days.

Test Series C1 (b) and C2 (b) are apparently the only tests made in which concrete was tested by keeping wet for varying periods and then by air drying until the time of test. The results of tests Series C1 (b) and C2 (b) are in general accord with the California Highway Commission tests[1] on cores drilled from field-cured concretes in 1925 and also with the results of a rather elaborate curing series made by the California Highway Commission and the Structural Materials Research Laboratory in 1924.[2] As a result of these tests, the conclusion was drawn that maximum 28-day strength could be had under unfavorable conditions of high temperature and low humidity by keeping specimens damp from 7 to 14 days. The conclusion on p. 33 of Prof. Gilkey's paper that specimens allowed to dry after 7 days will have only 65 or 70 per cent of the strength of standard 28-day specimens, would seem in view of the data given in Table 11 of Prof. Gilkey's paper and of the writer's experience and observation, to apply to small mortar specimens rather than to concrete in larger test specimens or in ordinary pavement or structures.

[1]Curing of Concrete, Recent California Experiments by C. L. McKesson, *California Highways*, January, 1926.

[2]Studies of Curing Concrete in a Semi-Arid Climate, Bulletin 15, Structural Materials Research Laboratory, Lewis Institute.

Watering of concrete and delayed use due to long curing periods is expensive and the watering and curing period should not, therefore, be extended beyond that which is necessary to give strengths required having in mind the proper factors of safety.

Mr. Gilkey.

HERBERT J. GILKEY (*By Letter*).—It is probable that the points mentioned by Mr. McKesson, who is himself an investigator of curing conditions, are such as might occur to many careful readers of this paper. It therefore seems worth while and desirable to give them rather detailed attention.

Briefly, the points made, or objections raised are as follows:

1. The majority of the tests were made on 2 x 4-in. mortar specimens.

2. His experience has not indicated that results from small mortar specimens are indicative of or parallel to results obtained from larger 6 x 12-in. specimens or mass concrete in the field.

3. Special exception taken to question No. 1 of the "Conclusion," "that specimens dry after 7 days will develop only 65 or 70 per cent of the standard 28-day strength of the wet cured specimen."

4. From the standpoint of economy the curing period should be as short as possible.

These majority objections will be considered in the order stated.

1. The majority of these tests were on the small specimens, as stated by Mr. McKesson, and the Universal Curing Diagram (Fig. 13) is based entirely upon 2 x 4-in. mortar series. Several attempts have been made to emphasize this fact. (See notes under the diagram, and footnote under Question 1 of the conclusions.) I hope that Mr. McKesson's re-emphasis of this point will prevent its being overlooked by others. However, Series C1, C2 and C3 are all of concretes (6 x 12 in.) and extensive tests of Abrams (a) and (b) and of Green are cited (See Tables I and II and Figs. 6, 7, 8, 11, and 12.) About one-sixth of these tests were on 6 x 12-in. concrete specimens while several hundred tests are represented by the work of Abrams and Green.

2. These tests all lead to the conclusion that results on the smaller mortar specimens are exactly parallel to those obtained from mass concrete or larger specimens. There is a difference in degree and rate but none in kind. The smaller specimen will respond more rapidly to any change in curing environment, just as Mr. McKesson has stated. This is shown very clearly in Figs. 6 and 11. The answers to Questions 7, 8, 9 and 10 of the conclusions consider in some detail the effects of water ratio, size, mixture, etc. The detailed discussion of Fig. 11 calls attention to the remarkable similarity of action between the concretes and mortars shown and also to the difference in extent and rate of action due to differences in size of specimens and atmospheric environments.

That the results of the mortar tests are not only indicative of the nature of the effects on concretes, but that commercial concretes may be affected by too short a curing period far more drastically than were any of the mortars reported here, is indicated by the tests of Abrams as plotted

on Fig. 12. The tabulation below is of the data from Table IV, Lewis Institute Bulletin No. 2, part of which is shown on Fig. 12, of this paper.

Effect of Different Periods of Moist Storage on Compressive Strength expressed as Percentage of Compressive Strength of Full Time Moist Stored Specimen of Same Test Age (Table IV, Bulletin No. 2, Lewis Institute).

Curve	Series	Material	W/C	Age at Test	Exposed to Air of Laboratory after Moist Storage of (in days) 0 (Dry)	3	7	14	21	Remarks
A	M1(e)(f)	Mortar	0.70	3mo.	45 (30)	70 (55)	78 (63)	95 (80)	108 (93)	
B	Abrams(a)	Concrete	0.66	4mo.	44 (29)	56 (41)	72 (57)	85 (70)	105 (90)	
—	"	"	0.73	"	42 (27)	58 (43)	.. ..	.. ..	92 (77)	Not plotted on Fig. 12.
C	"	"	0.81	"	45 (30)	50 (35)	59 (44)	75 (60)	86 (71)	
—	"	"	0.91	"	37 (22)	46 (31)	.. ..	.. ..	87 (72)	Not plotted on Fig. 12.
D	"	"	0.99	"	36 (21)	36 (21)	44 (29)	60 (45)	77 (62)	
E	"	"	1.09	"	35 (20)	33 (18)	40 (25)	53 (38)	65 (50)	

It should be noted that Fig. 12 gives the age *after* removal from mold and that the actual age (which is used in the above tabulation) is one day greater than indicated by the horizontal scale of Fig. 12. Thus the data for Cols. 3, 7, 14 and 21 of this table would be found at 2, 6, 13 and 20 days in Fig. 12. The figure in parenthesis indicates the relative strength if the specimens compared be in the same condition at test. The specimens on which the percentages are based were all stored moist for the full test age and tested moist. The specimens removed from storage were all dry at test. An arbitrary figure of 15 per cent has been assumed as representating the gain in strength by virtue of the dried condition (or loss in strength of the standard specimen by virtue of being wet at test). As elsewhere noted, see question 6 of conclusions, this difference may actually be any where from 10 to 40 per cent, the 15 per cent assumed being well toward the lower limit. It is probable that the time difference between being dry and moist at test would vary considerably for the wide range of water-cement ratios covered but it is doubtful if the difference would be less than 15 per cent in any of these cases.

In comparing Abrams' figures with those of the Colorado tests, several points are brought out.

(a) On the basis of 4-month strengths, the effect of an incomplete curing is more pronounced than on the basis of 28-day strength tests. This accords fully with the findings stated in Question 2 of the conclusions and shown on Fig. 13. Gain of strength ceases when the concrete or mortar becomes dry while that remaining moist will cure and take on strength indefinitely or practically so, at a constantly diminishing rate.

(b) The fact that the premature removal is much more detrimental to concretes having high water-cement ratios accords with the conclusion of Question 7.

(c) The mixtures of practice are rarely those of water-cement ratios below 0.80 and are usually those with ratios from 1.00 to 1.10. For these mixtures Abrams' tests show that the effect of unfavorable curing is even greater for concrete 6 x 12-in. cylinders than it is for mortar 2 x 4-in. specimens of our tests. These data would seem to indicate that the relatively dry mixture used in the 8 x 4-in. mortar specimens more than offsets any advantage that the larger concrete specimens might have by virtue of size (slower response to an unfavorable environment) and possible greater range of fluctuation for mortars than concretes (mentioned in Question 9 of the conclusions).

3. The answer to Question 1, of the conclusions was derived from Fig. 13. Footnotes under both Question 1 and Fig. 13 specifically state that such citations are from small mortar test specimens and are illustrative only. Nevertheless the results of Abrams (Fig. 12 and the tabulation given above) indicate very clearly that for usual mixtures "dry after 7 days" is likely to produce a *concrete* weaker than 65 or 70 per cent of the standard, rather than stronger. Moreover the difference becomes greater with added age rather than less, as noted under 2, (unless favorable curing conditions be again attained).

It is probable that there is really no disagreement between Mr. McKesson's experience and the results of these tests. A specimen dry after 14 days, tested dry might well approximate, or even exceed the 28-day strength of a standard moist cured specimen tested moist. But that same moist specimen has a dry strength from 10 to 40 per cent above its wet-tested strength. Mr. McKesson has probably not been thinking in terms of specimens that were in the same condition at test. Nor are results of flexural tests necessarily parallel to those from compressive tests.

4. Watering of concrete and delayed use due to long curing periods is expensive and it is not the intention to here recommend, by inference or otherwise, that unnecessary expense be incurred in order to attain maximum strength prior to use. Especially is such procedure not necessary in the case of pavements. A pavement may be safely placed in service just as soon as the combination of curing and hardening by drying out give it strength enough to stand up under the traffic. At some seasons of the year the sub-base will again contain moisture and curing will resume. This resumed curing will develop strength more slowly than the original curing (as noted in the discussion of question 3 of the summary) but tests made after this paper was prepared seem to indicate quite definitely that the specimen with a poor start or having an interrupted curing period will eventually "catch up" if later favorable storage conditions be continued for a sufficient length of time. As stated in the footnote to the summary (p. 427) it is often true that cores drilled from pavements prove to be as strong or stronger than samples of the same concrete cured under almost ideal laboratory conditions. In nearly all locations there is present in the subgrade during several months of the year enough moisture to satisfy all curing needs. As noted in question 13 of summary and in

footnote mentioned above this condition is not true for concrete in buildings and greater expense is there justified in order that the concrete approximate its design strength before moist curing is stopped. In this latter case, it will often be more economical to use a richer mix for the development of high early strength in preference to a too long wetting down period or leaving on of forms.

Relation of 7-Day to 28-Day Compressive Strength of Mortar and Concrete.*

By W. A. Slater.**

There seems to be a widespread belief that there is little relation between the strength of concrete at an age of 7 days and that at 28 days. This belief stands in the way of using the 7-day strength as a criterion for determining whether concrete on construction work will acquire at a later date the strength required by the design.

Results of the field tests carried out at Camden, N. J., and Newark Bay, N. J., under the sponsorship of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[1] suggested a fairly definite relation between 7-day strength and 28-day strength. This indication has been confirmed by a study of results of laboratory tests of concrete and mortar. The accompanying figures show the 7-day strengths plotted as abscissas and the 28-day strengths as ordinates. The curved line shown in each diagram is the graph of the equation

$$f'_c = f + 30\sqrt{f} \qquad \text{(1)}$$

This equation represents fairly well the relation between the 7-day strength f, and the 28-day strength, f'_c.

Most of the data are taken from the publications of the Structural Materials Research Laboratory, Lewis Institute, Chicago, but enough are from other sources to indicate that the relation pointed out is found to apply to other data.

In most cases the strengths reported represent the average of five tests of a kind for both the 7-day and the 28-day ages.

The exceptions were as follows:

Fig. No.	No. of specimens of the same kind
1	4
2 (b)	4
3 (Tables 20, 21, 28 and 30)	3
3 (Tables 31B and 31C)	2
4 (b)	24 or 60
10 (a)	Unknown
11 (1: 5.2 mix)	4 to 10
11 (1: 2.5 mix)	5
12	5
15	3
16	3

* Published by permission of the Director of the Bureau of Standards.
**Engineer-Physicist, Bureau of Standards, Washington, D. C.
[1] See Am. Soc. C. E. *Proceedings*, January, 1925.

Where admixtures with the cement were used the material was generally added, in the percentages by volume indicated in the diagrams, to a fixed quantity of cement, that is, the percentage of admixture is given as a percentage of cement. In Fig. 9d, however, the admixture replaced a corresponding quantity of cement and is stated as a percentage of cement plus admixture. In Fig. 6 the percentage of tannic acid gives the concentration of tannic acid in terms of the weight of the mixing water.

Figs. 1 and 2 give the strengths of concretes made from different materials, from different proportions of the same materials, and with different water-ratios when the proportions were kept the same. In all cases the relation between the 7-day and the 28-day strength is fairly well represented by Equation (1).

In Fig. 3 are data from Technologic Paper No. 58 of the Bureau of Standards. A few points show 28-day strengths too low to fit Equation (1). Of these one group of specimens was steam cured and since steam curing accelerates the hardening it is to be expected that the relation between the 7-day and the 28-day strength would be disturbed. Three other points fall too low for the equation.

In Fig. 4c the 28-day strength for concretes made from cements of varying degrees of fineness were consistently a little lower than the strengths indicated by the equation. The variation was not greater than must be expected in any case where experimental results are involved.

Notwithstanding the deterioration of the cement with age and the harmful effects of tannic acid as shown by Bulletins 6 and 7 respectively of the Structural Materials Research Laboratory the relation of the 28-day to the 7-day strength is shown quite satisfactorily in Figs. 5 and 6 by means of Equation (1).

Figs. 7, 8, 9 and 10 are used to show the effect of a variety of powdered admixtures used in different percentages, on the compressive strength of concrete. All the data except those of Fig. 10a are from Bulletin 8, Structural Materials Research Laboratory. With the possible exception of calcium chloride no effect of the admixtures on the relation between the 7 and 28-day strengths can be seen. It is not certain whether the wide variation of the strength in Fig. 10a from the curve shown represents an effect of calcium chloride or whether it is merely a result of erratic test data. The points for zero calcium chloride when plotted by themselves show an agreement with the curve which is only slightly less satisfactory than that found in other curves of this paper. When the points shown in Fig. 10a are segregated into groups according to the percentage of calcium chloride there is some indication that the higher the percentage of calcium chloride the higher the points lie with reference to the curve.

Figs. 11, 12 and 13 have been plotted from Bulletin 13 of the Structural Materials Research Laboratory to show the effect of calcium chloride admixture on the relation between the 7-day and the 28-day strengths. Fig. 11 includes 1: 5.2 and 1: 2.5 concretes with admixtures in varying amounts, of calcium chloride in five different forms. Fig. 12 includes pro-

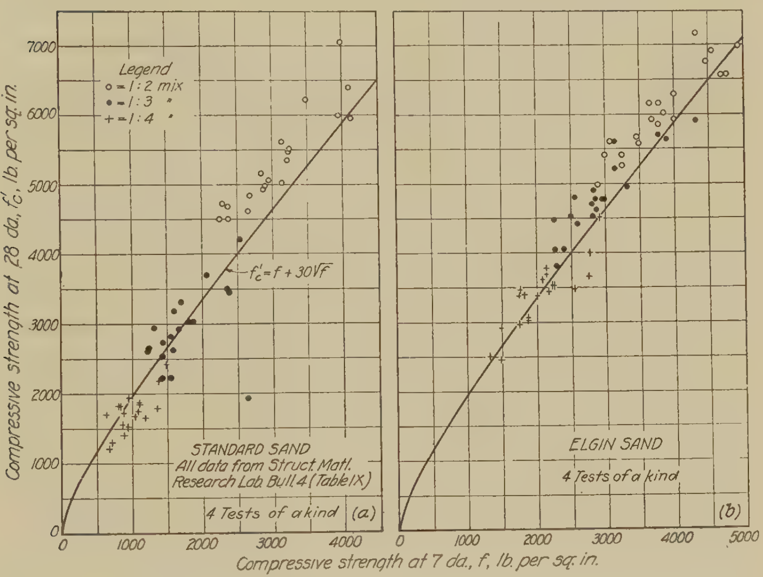

FIG. 1.—COMPRESSIVE STRENGTH OF 2 X 4-IN. MORTAR CYLINDERS WITH VARYING PROPORTIONS OF CEMENT.

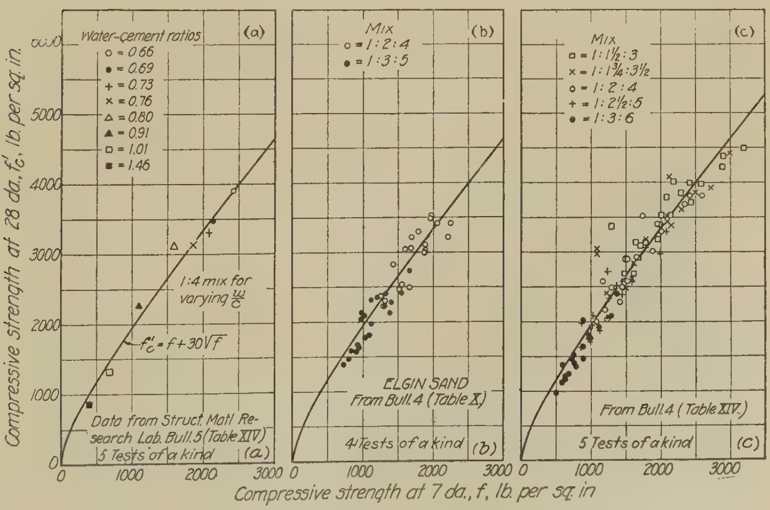

FIG. 2.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING WATER-CEMENT RATIO AND VARYING PROPORTIONS.

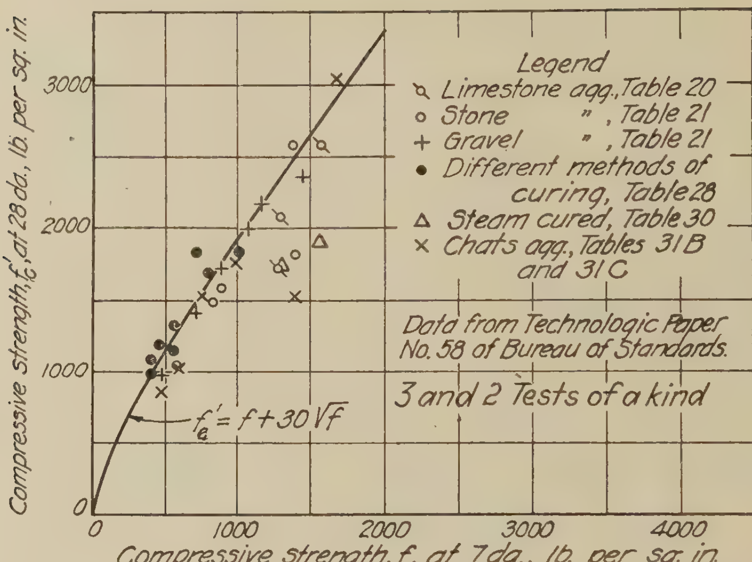

FIG. 3.—COMPRESSIVE STRENGTH OF MISCELLANEOUS 8 X 16-IN. CYLINDERS.

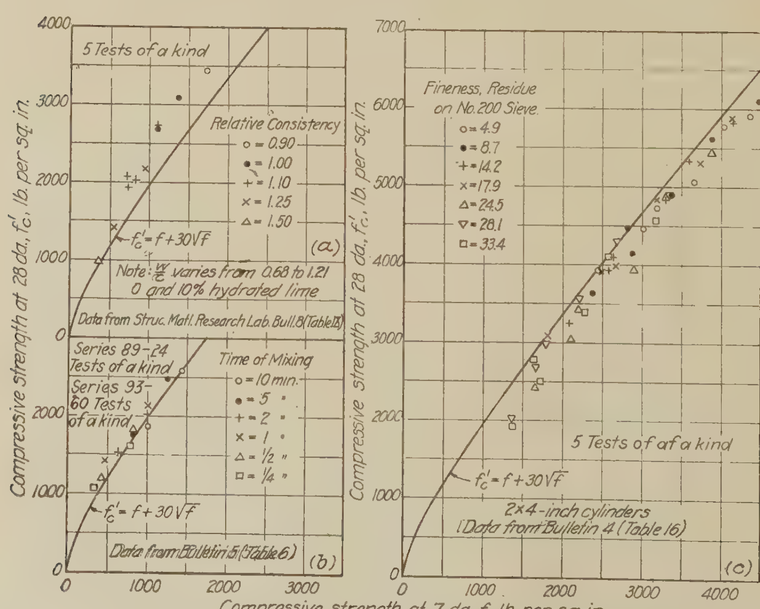

FIG. 4.—COMPRESSIVE STRENGTH OF 2 X 4-IN. AND 6 X 12-IN. CYLINDERS WITH VARYING FINENESS OF CEMENT, CONSISTENCY AND TIME OF MIXING.

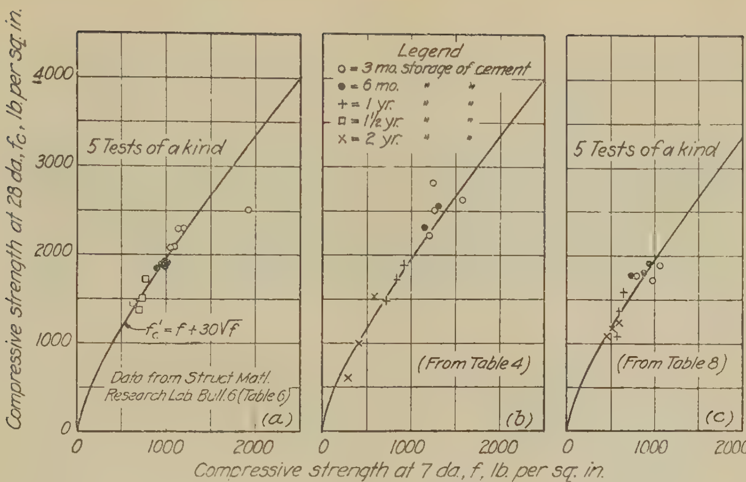

FIG. 5.—EFFECT OF STORAGE OF CEMENT BEFORE USE IN CONCRETE; 6 x 12-IN. CYLINDERS.

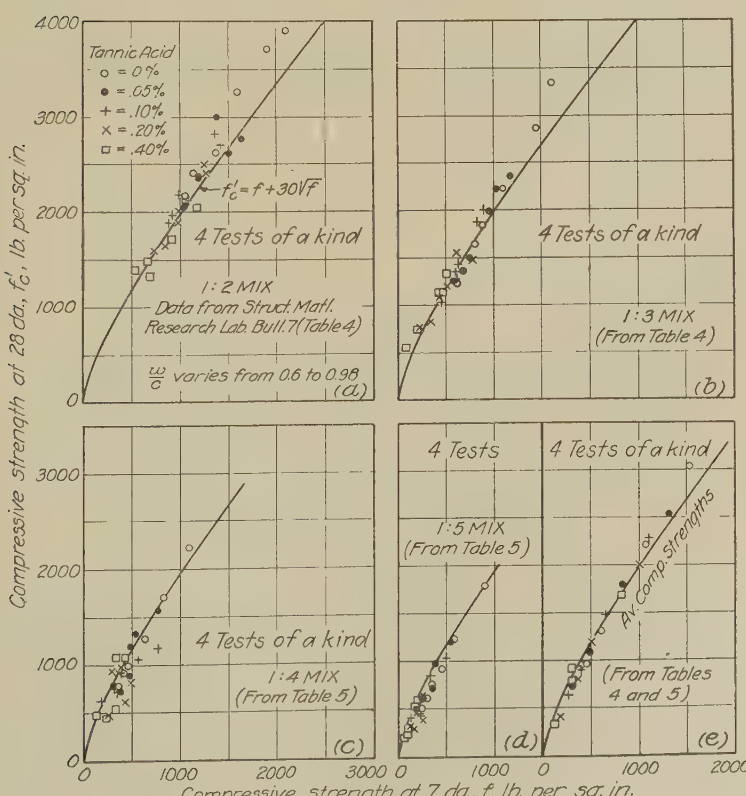

FIG. 6.—COMPRESSIVE STRENGTH OF 3 x 6-IN. CYLINDERS WITH VARYING PERCENTAGES OF TANNIC ACID.

portions of cement to aggregate of 1:7, 1:4, 1:3 and 1:2. In these figures more markedly than in Fig. 10a it appears that the addition of calcium chloride had the effect of causing a smaller proportionate increase of strength than is found when no calcium chloride is present. This result may be accounted for by the fact that, as shown in Bulletin 13, the proportionate increase in the 7-day strength due to the use of calcium chloride was considerably greater than the proportionate increase in the 28-day strength.

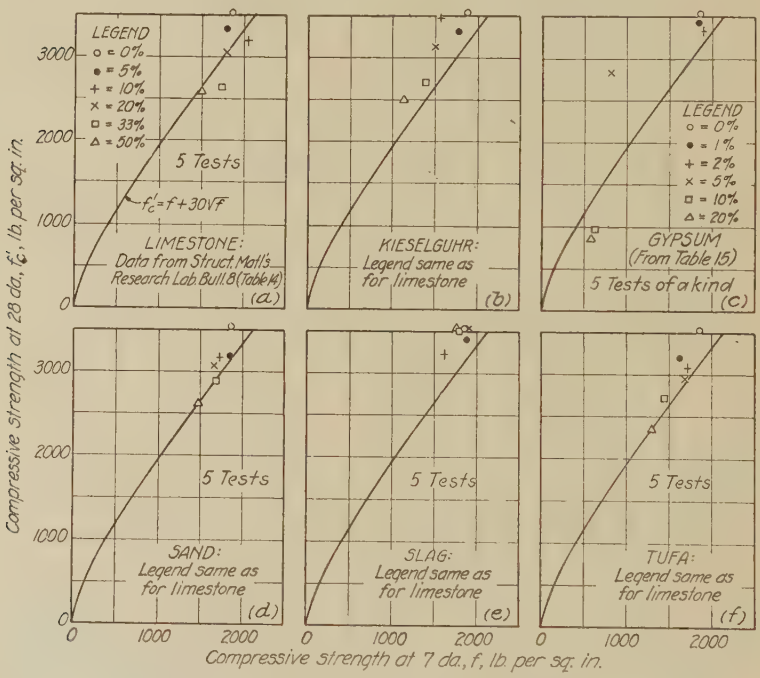

FIG. 7.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF POWDERED ADMIXTURE.

Fig. 13 has been prepared to determine the effect of curing conditions on the relation of the 7-day to the 28-day strength. This figure shows that the curing conditions have an important effect on the relation and indicate that in order for Equation (1) to apply the storage should be under moist conditions throughout the curing period.

The variation in storage conditions affected the average 7-day strengths shown in Fig. 13 less than it affected the average 28-day strength. This fact is favorable to the use of the 7-day test as a gage of the 28-day strength of the concrete when cured in moist air or sand.

In Fig. 14 the strengths for 2 x 4-in. mortar cylinders are given. The points for specimens made from standard Ottawa sand on two different dates lie close to the curve. The points for specimens made from Elgin sand lie slightly above it. The agreement on the whole is satisfactory.

In Bulletin No. 10 of the Maine Technology Experiment Station "The Prediction of the 28-Day Breaking Strengths of Mortars from Their 7-Day Results," by Gowen, Leavitt and Evans, the equation

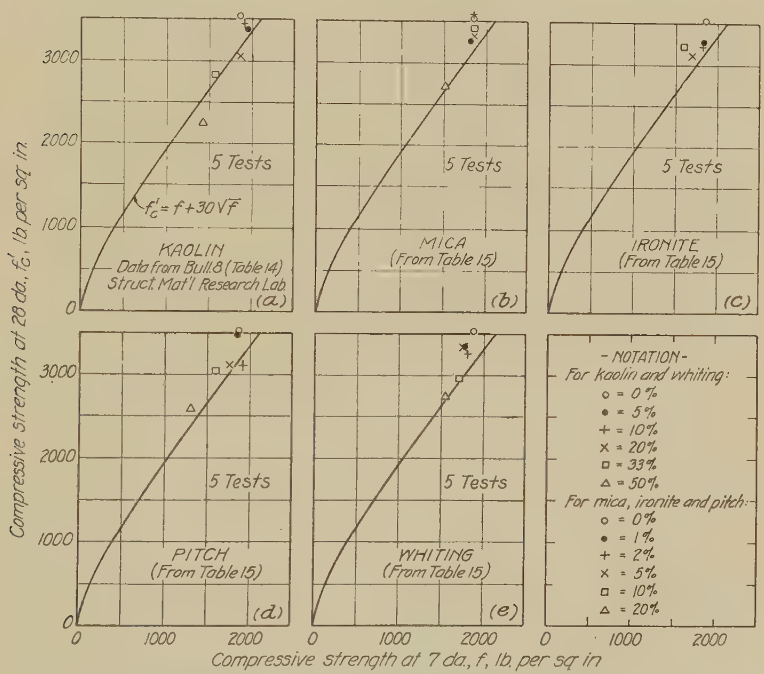

FIG. 8.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF POWDERED ADMIXTURE.

$$f'_c = 793 + 1.128 f \qquad (2)$$

is shown to fit the results of 166 tests of 2 x 4-in. mortar cylinders of a 1 : 2 mix. The agreement of this equation with the results from which it was derived is close, and it seems that the difference between those test results and the results here reported is measured approximately by the difference between Equations 1 and 2. The University of Maine tests, however, include no specimens with a 7-day strength less than 1,200 lb. per sq. in. and it sems likely that the straight-line equation would not have

been found to fit the test results for low strengths. For comparison the graph of equation (2) is shown in Fig. 14.

The straight-line equation

$$f'_c = 633 + 1.32 f \quad \text{.......................} (3)$$

whose graph is shown in Fig. 14 gives the same 28-day strengths as Equa-

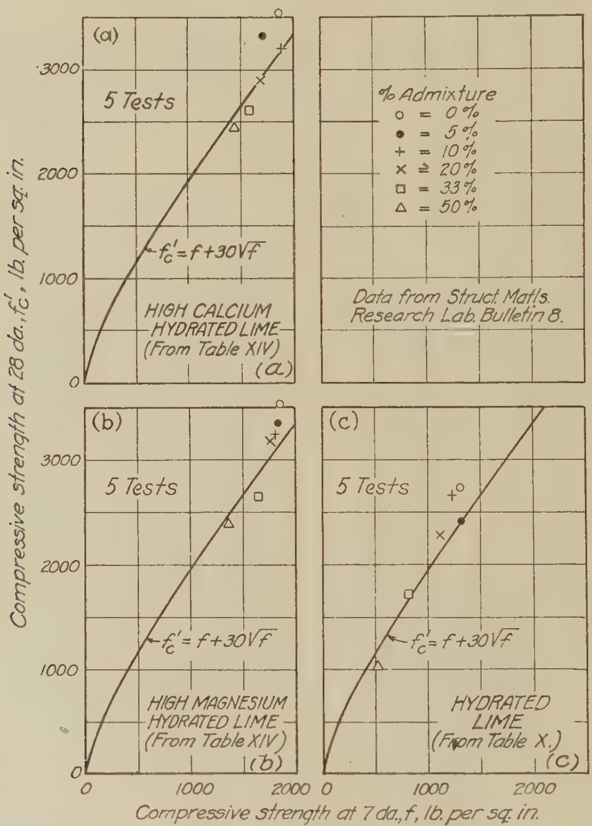

FIG. 9.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF HYDRATED LIME.

tion (1) for 7-day strengths of 1,000 and 4,000 lb. per sq. in. and for strengths between these values fits the test results with sufficient accuracy. This equation is somewhat simpler than Equation (1) but does not apply for 7-day strengths below 1,000 lb. per sq. in.

Results of tension tests are shown in Fig. 15. It is clear from this

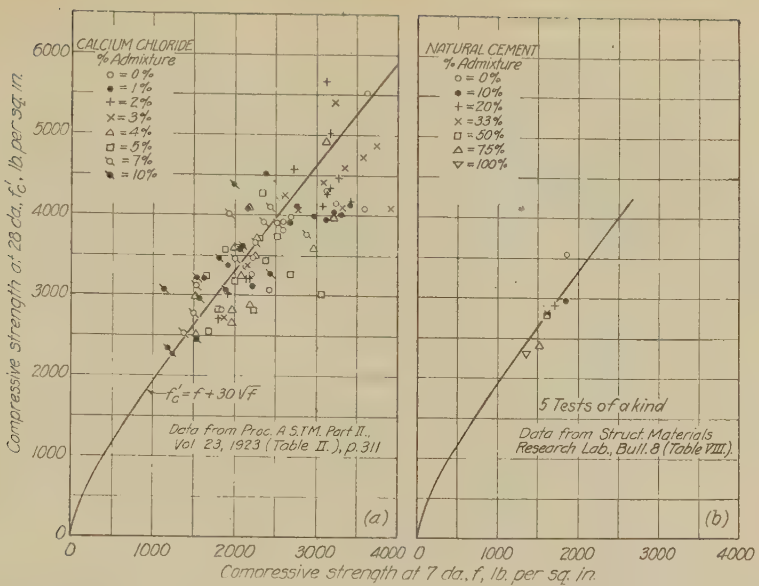

FIG. 10.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF CALCIUM CHLORIDE OR NATURAL CEMENT ADMIXTURES.

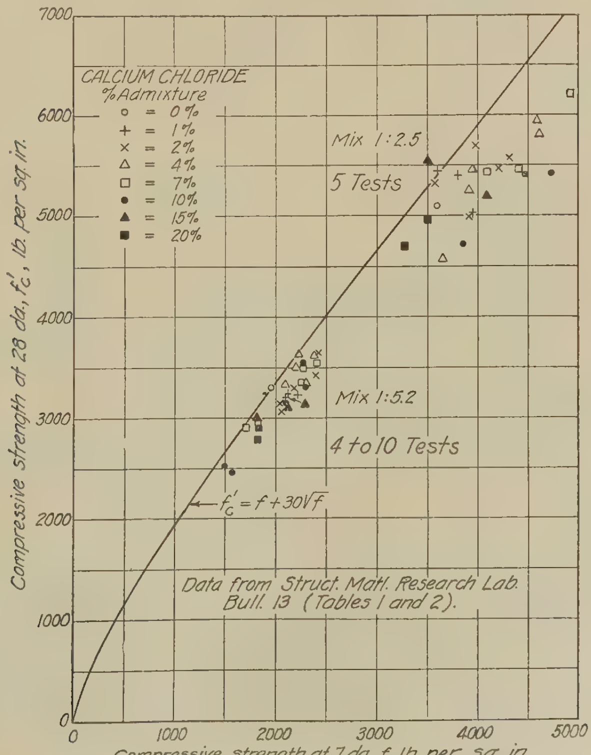

FIG. 11.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF CALCIUM CHLORIDE; MIX 1:5.2, ANND 1:2.5.

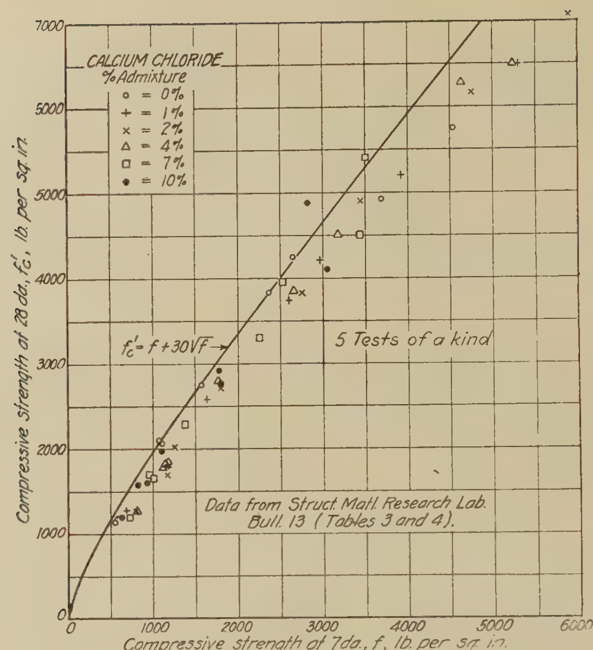

FIG. 12.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH VARYING PERCENTAGES OF CALCIUM CHLORIDE; MIX 1 : 7, 1 : 4, 1 : 3 AND 1 : 2.

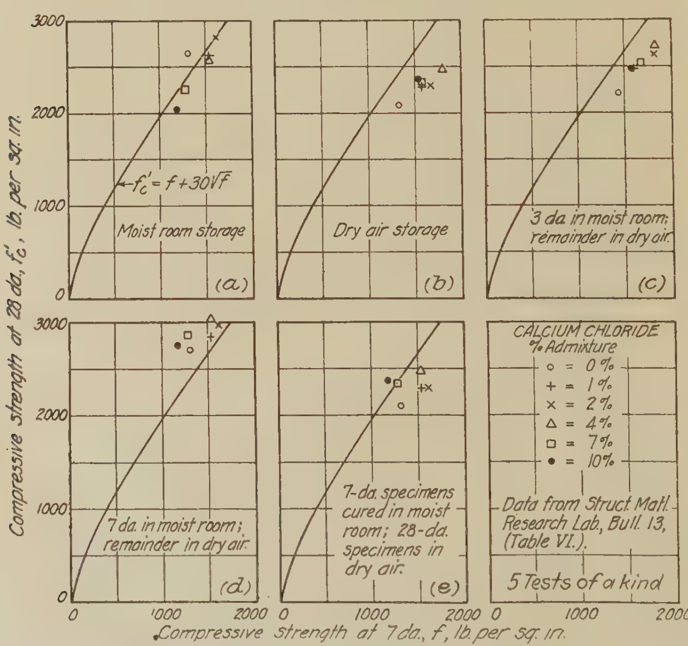

FIG. 13.—COMPRESSIVE STRENGTH OF 6 X 12-IN. CYLINDERS WITH CALCIUM CHLORIDE ADMIXTURES CURED UNDER VARYING CONDITIONS.

figure that the proportionate increase of the tensile strength between the 7 and 28-day ages was much less than that of the compressive strength.

Fig. 16 is reproduced from the report of the field tests carried out at Camden and Newark Bay, N. J., under the sponsorship of the Joint Committee. (See Am. Soc. C. E. *Proceedings*, January, 1925, p. 3.) Each

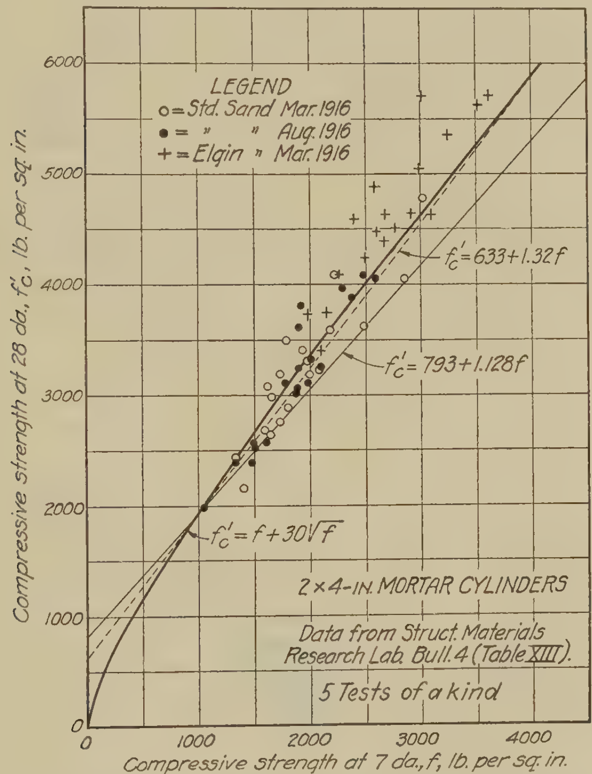

FIG. 14.—COMPRESSIVE STRENGTH OF 2 X 4-IN. MORTAR CYLINDERS OF 1:3 MIX; CEMENT PASSING NO. 200 SIEVE VARIES FROM 2.2 PER CENT TO 34 PER CENT.

point represents the average of three 7-day strengths and three 28-day strengths. Approximately 80 per cent of the points show 28-day strengths varying not more than 15 per cent from the strength shown by Equation (1). The other 20 per cent show variations of from 15 to a maximum of 40 per cent. For the laboratory tests, Figs. 1 to 14, most of the 28-day strengths show a smaller variation than this from the strength given by the equation. In a few cases such as Figs. 11, 12 and 13 the entire group

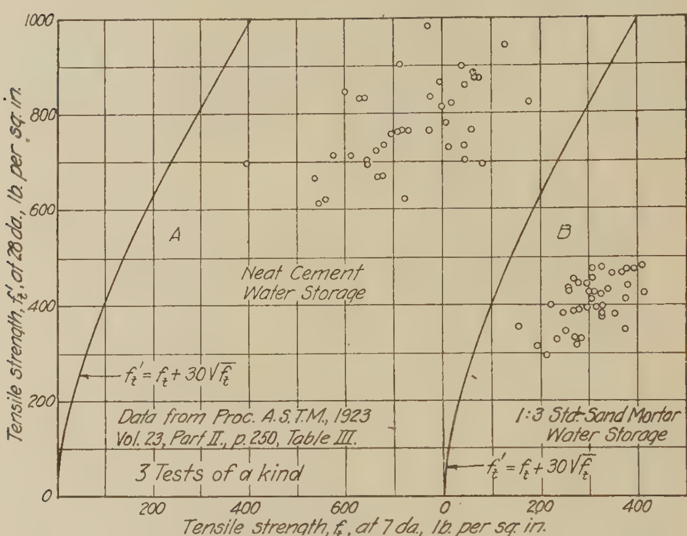

FIG. 15.—TENSILE TEST OF STANDARD BRIQUETS OF NEAT CEMENT AND 1:3 MORTAR.

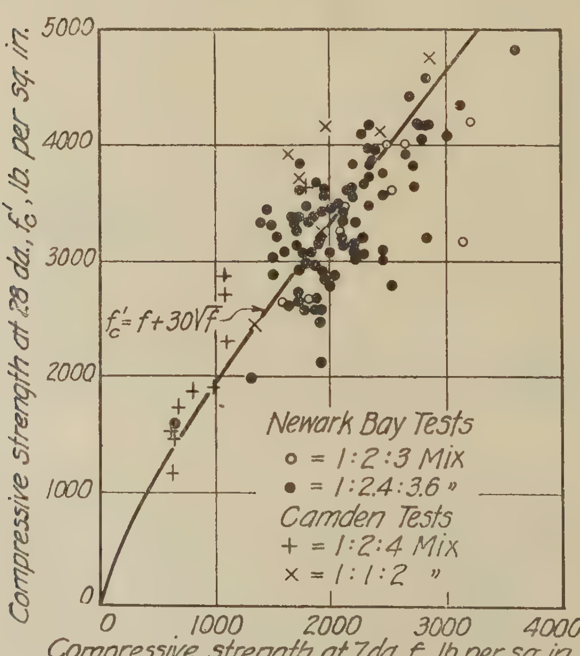

FIG. 16.—RESULTS OF FIELD TESTS AT CAMDEN AND NEWARK BAY, N. J., FOR JOINT COMMITTEE.

of points appears to lie above or below the graph of the equation, indicating a systematic rather than an accidental variation.

The data included in Figs. 1 to 16 involve many of the variables which may affect the compressive strength of concrete. Yet the results are sufficiently concordant to warrant the conclusions that within the limits of accuracy required on construction work, the 28-day compressive strength of concrete may be predicted from the 7-day compressive strength by means of Equation (1).

If Equation (1) is to be used to predict the 28-day strength of field specimens tested at 7 days it will be important that the field specimens be placed in damp sand as soon as possible after they are made in order to avoid the effect of variations in the early storage conditions on the 7-day strength.

The use of Equation (1) for predicting the 28-day strength would not remove the necessity of making 28-day tests before the beginning of construction as a gage of the suitability of materials and proportions proposed for use.

DISCUSSION.

Mr. Schnarr. WILFRED SCHNARR (*By Letter*).—The Hydro-Electric Power Commission of Ontario in its field work has used 7-day compressive tests as an indicator of probable 28-day strengths and has thus accumulated considerable data on the relation between the two. These data in the form of charts are shown herewith.

Fig. 1, 2 and 3 cover different field jobs and are representative of the type of data that have been obtained. Fig. 4 is from tests made in the Toronto laboratory of the commission. In the field tests, the 7-day results are from an individual specimen; the 28-day are an average of two. In the laboratory tests both 7-day and 28-day results are an average of 5 specimens. Damp sand curing is used in the field; moisture saturated air in the laboratory. Details of the materials and conditions on the three field jobs are given in Mr. Young's paper. (See p. —.)

It is noticeable that only in one of these curves do the data agree with the theoretical curve. In Figs. 1 and 2 the results are below, in 3 and 4 the slope of the average line is steeper than for the theoretical curves of Professor Slater. These curves are so typical of many others which have been observed by the commission, both in field and laboratory tests, that these divergences would seem to have some common cause and therefore the conditions under which the different tests were made have been carefully analyzed.

The factor which seemed to best explain the differences in the curves was the different rates of the setting of the cement.

Four cements are represented, A, B, C and D, and on some jobs, two of these were used. Cement C has a rather high 7-day strength and in Fig. 1 the concrete tests fall considerably below the line. The cement in Fig. 2 was for the most part D, with the balance B. D has a fair strength for the 7-day test while B has a very high 7-day test. The tests for D are close to the curve, while B which had the high 7-day strength is considerably below the line.

In Fig. 3 the cement used was divided about equally between brands A and B. Cement A has a normal 7-day strength and the 28-day strength somewhat higher than the average. This accounts for the group on the upper side of the curves, while those close and a little below are cement C, which in this case seems to have a relation that is more comparable to Professor Slater's tests than the rest. In Fig. 4, the laboratory tests, cement A was used, and half of these tests and especially those of the higher strength lie above Slater's curve.

The aggregate or the curing condition seem to have had little influence on the results obtained. The aggregate used on each job was a different

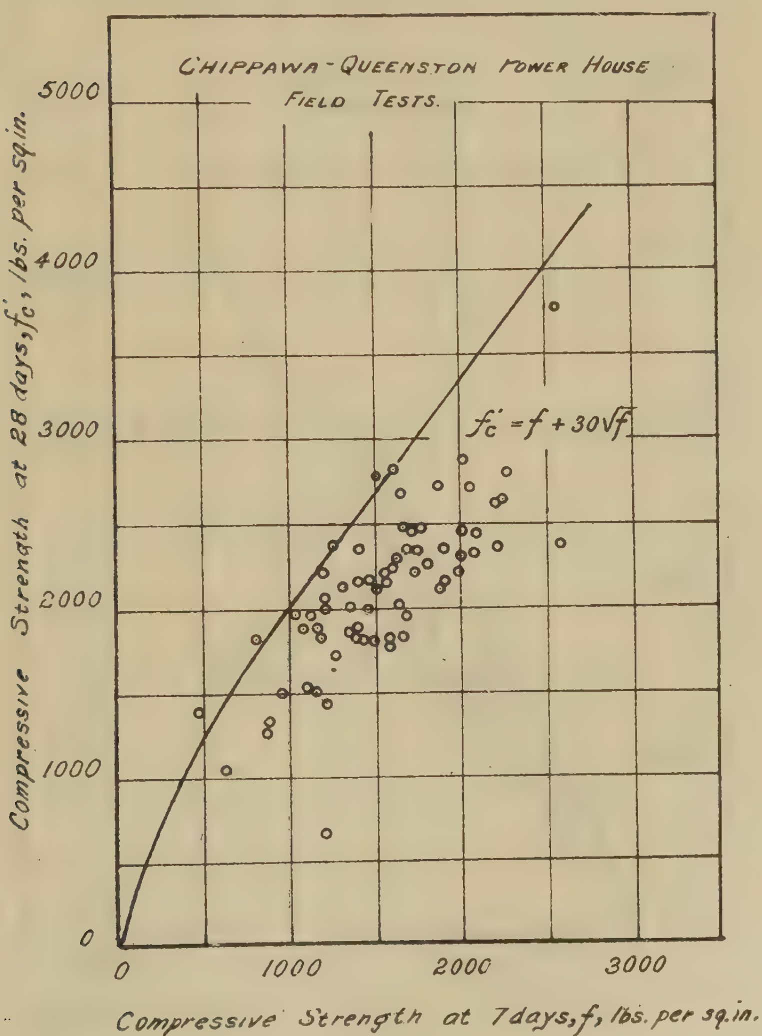

FIG. 1

material, but for any individual job, the aggregate was very uniform. The specimens were cured in the usual manner, stored in damp sand or in a

moisture saturated air and were tested on the job, with the exception of the tests of Fig. 2 where no testing machine was available in the field. In this

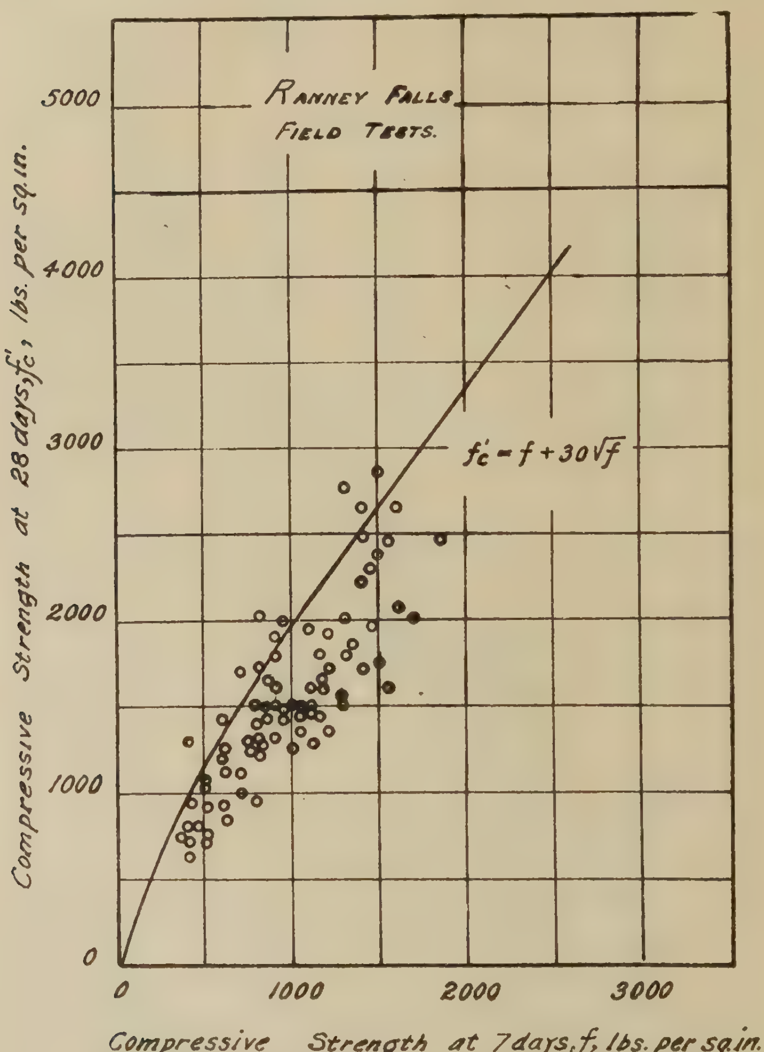

FIG. 2

case all cylinders were stored at the works and shipped in boxes containing moist sawdust to the Toronto laboratory, where they were stored until tested.

The results as shown by the four charts would indicate a wide range of strength, at 28 days for any one value at 7 days, approximately 1000 lb., and for this reason we would not be inclined to depend too greatly on the

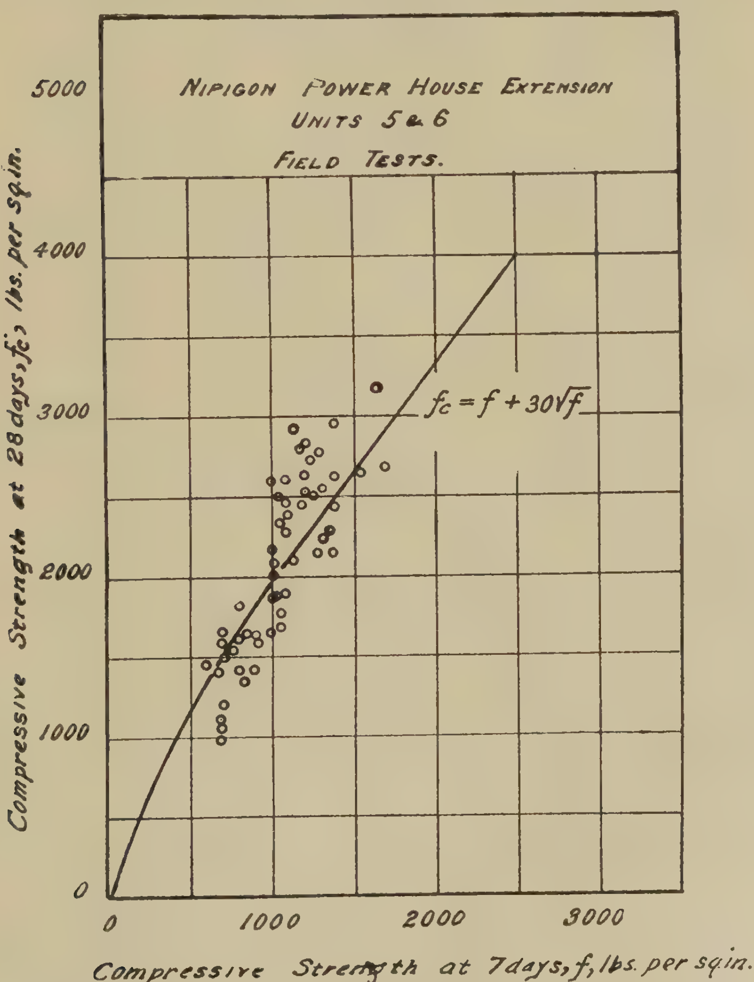

FIG. 3

earlier tests, for indicating the 28-day strength. Again using the 7-day tests tends to give a preference to high early setting cement, and such cements are not necessarily better than those that set up more slowly.

On the strength of these data which are typical of other data in the possession of the commission, we are inclined to believe that it would be

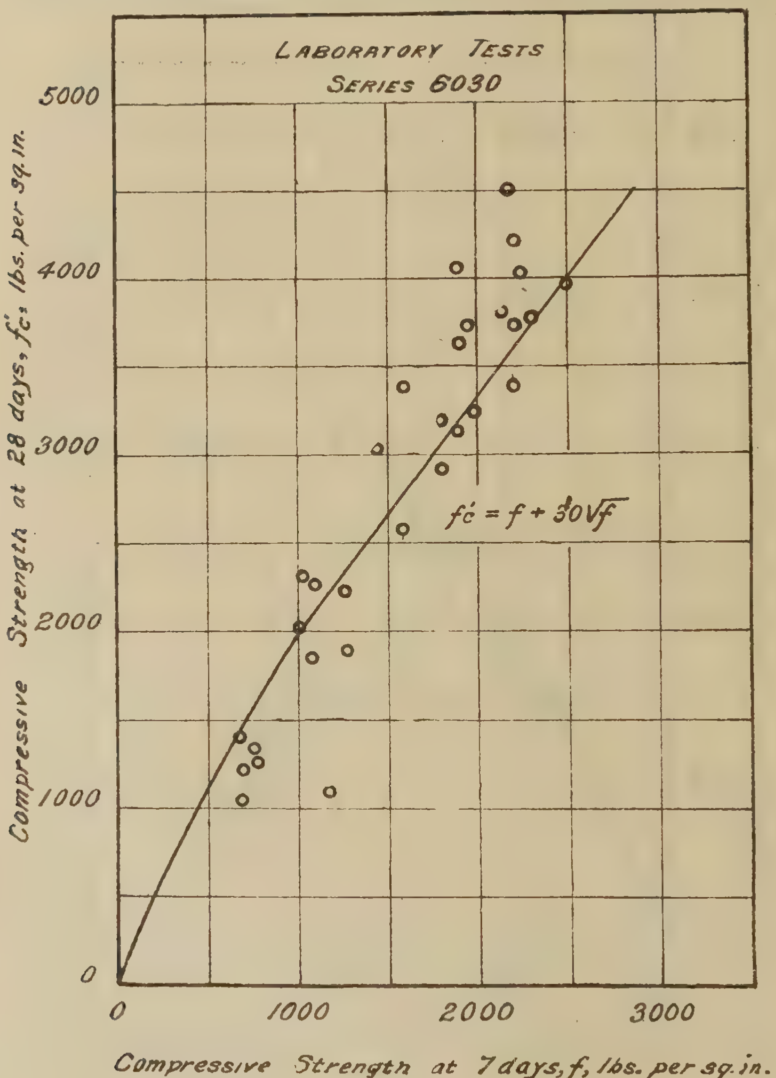

FIG. 4

well to consider the possible effect of curing and cement in predicting 28-day strength from 7-day tests, as they have a direct bearing on the relationship between the two.

JOHN W. GOWEN AND H. WALTER LEAVITT (*By Letter*).—The problem of the relation between the 7-day and 28-day test on mortars[1] has interested us for some time. In fact, we were perhaps the first laboratory to seriously attempt its solution. While many engineers were yet arguing that the 7-day test was either of no value or very doubtful value it was possible to show on more material than has as yet been analyzed elsewhere that the 7-day mortar strength has a fairly high relation to its 28-day strength. **Mr Gowen. Mr. Leavitt.**

Mr. Slater's results differ from ours in detail only. He uses an exponential equation in place of our linear equation to describe the relation between the 7-day and 28-day test results. It is of interest to examine the reasons for choosing linear equations for describing the results on mortar or concrete strength. Linear equations were chosen because of the fact that so far as engineering experience has shown and so far as our data were able to show, the results throughout supported the assumption of a linear relationship. This conclusion was supported by 478 tension tests (3 briquettes each, 1: 3 mix) and 166 compression tests (3 cylinders each, 1: 2 mix) on natural sands of the state of Maine. Since our original study was completed these data have been much extended, in fact, our subsequent 254 tension tests and 251 compression tests support the same conclusion, namely, that the relation between the 7-day and the 28-day tests are linear in so far as practical engineering experience will allow us to determine.

Mr. Slater has attempted to apply his equation to mortar, cement or concrete, made under many different conditions and mixes, whereas in our work we attempted to apply our equation only to our standard mix of 1: 2 for compression[2] and 1: 3 for tension. It is believed that this further extension is of doubtful value because of the fact that in studying the relation between strength increases to increases in the cement sand ratio we have found that the relation becomes non-linear. It is therefore introducing a slightly different variable to apply an equation to all materials when it is also to be applied to sands of a single mix. Furthermore it was possible to show that every laboratory has its personal equations.[3] This personal equation materially influences the relation between the 7-day and the 28-day test. It is, in our judgment, more desirable to find out the personal equation in the laboratory in making the prediction equations to be used in that laboratory than it is to attempt to make an equation which takes no account of this factor and consequently predicts poorly under many conditions.

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[1]See Gowen, John W. and Leavitt, H. Walter, "The Prediction of the 28-day Breaking Strength of Mortars from their 7-day Results." Maine Technology Experiment Station Bulletin No. 10, 1925.

[2]See Gowen, John W. and Leavitt, H. Walter, "The Choice of a Mix for Determining the Compressive Strength of Mortars." Maine Technology Experiment Station Bulletin No. 5, 1923.

[3]Gowen, John W. and Leavitt, H. Walter, "The Strength of Mortars in Relation to the Experience and Personal Equation of the Operator." Maine Technology Experiment Station Bulletin No. 11, 1925.

Mr. Slater presents data on thirteen different tables where the numbers are sufficient to have a meaning. Of these thirteen there are eight which

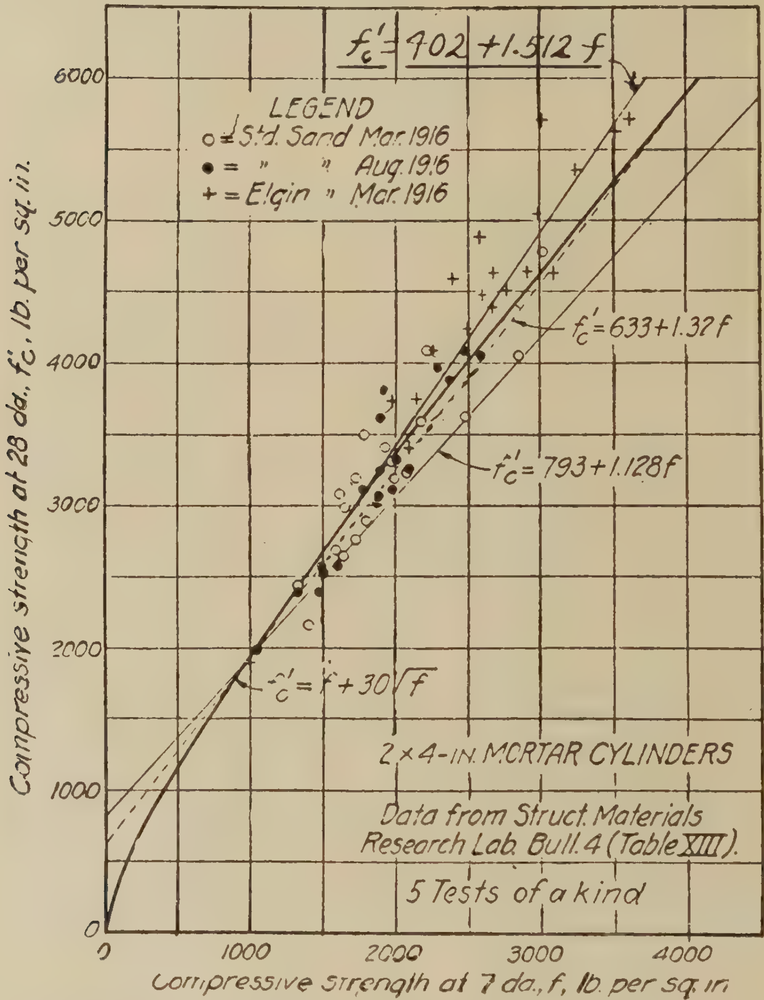

FIG. 1.—THE FIT OF EQUATION $f'_c = 402 + 1.512 f$ UPON MR. SLATER'S DATA OF FIGURE NO. 14.

[Gowen and Leavitt Discussion]

have a good fit and five which poorly fit. The data are in each table heterogeneous in one way or another either through the use of the different cements, through the use of different aggregates, the proportion of tannic

acid, proportions of calcium chloride or some such admixture. In general the data do not seem comparable with those with which engineering practice has to deal. In almost every case the data could be fitted more accu-

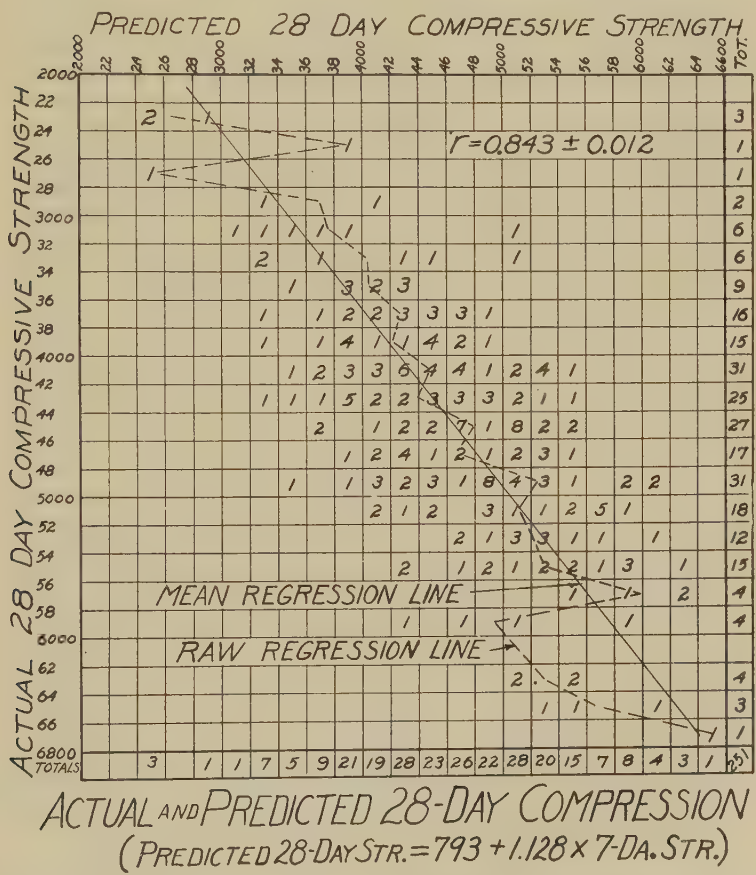

FIG. 2.—UNIVERSITY OF MAINE EQUATION APPLIED TO SUBSEQUENT TESTS ON 251 NATIVE MAINE SANDS.

[Gowen and Leavitt Discussion]

rately by the proper linear equation than by the exponential formula suggested.

Mr. Slater very promptly tries to apply the equation which we presented for our data on but one chart and even then there is considerable question as to whether or not the data he uses are suitable for this appli-

cation. As might perhaps be expected our equation fits rather poorly, his equation fits only fairly well. These data can, however, be fitted accurately by the same method by which our previous equations were derived. The resulting equation is

$$f^{1}_{c} = 402 + 1.512\, f$$

This equation accurately fits the results. It is a much better fitting equation than either of the other two as is shown in Fig. 1.

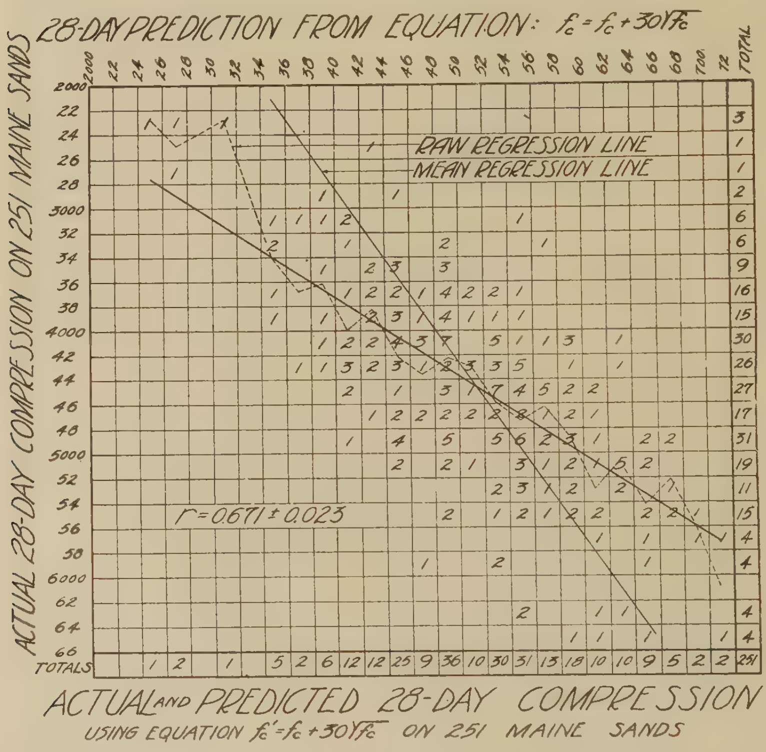

FIG. 3.—MR. SLATER'S EQUATION APPLIED TO SAME DATA AS THAT OF FIG. 2.

[Gowen and Leavitt Discussion]

At the University of Maine laboratory 251 tests for compressive strength at 7 and 28 days have been performed since our original study. These data are all on the 1 : 2 mix under carefully controlled conditions. They are made with nearly all types and kinds of native sands which the engineer meets in actual practice.

Fig. 2 shows the results of applying our prediction equation to these 251 samples. It is obvious that the prediction is on the whole fairly good

as the average actual test for each prediction agrees well with the predicted value. The measure of the accuracy of this prediction is given by the correlation coefficient between the actual 28-day compressive strength and the predicted 28-day compressive strength (the prediction being determined from the 7-day results). This correlation coefficient is found to be 0.843 ± 0.012. The agreement is consequently rather close between the 28-day compressive strength as determined from the 7-day compressive strength and the actual strength which the mortar was able to develop.

Fig. 3 shows the results of applying Mr. Slater's equation to the same data. The prediction is obviously not so accurate as that which is derived from the equation we previously presented.

If the correlation coefficient be determined it is found to be 0.671 ± 0.023 or much less than was obtained by the use of our prediction equation.

TABLE I.—ILLUSTRATING THE ACCURACY OF PREDICTION OF MR. SLATER'S EQUATION AND THE UNIVERSITY OF MAINE EQUATION UPON THE TEST RESULTS OF 251 NATIVE MAINE SANDS. THE DATA ARE THE SAME AS PRESENTED IN FIGS. 2 AND 3.

1	2	3	4	5	6
Actual Average 28-Day Compression Lb. Per Sq. In.	Predicted 28-Day Compressive Strength U. of M. Equation	Mr. Slater's Equation	Difference Between Col. 1 and Col. 2	Difference Between Col. 1 and Col. 3	No. of Sands
2,300	2,650	2,750	+350	+450	3
2,500	3,900	4,300	+1400	+1800	1
2,700	2,500	2,700	—200	0	1
2,900	3,700	4,200	+800	+1300	2
3,100	3,760	4,160	+660	+1060	6
3,300	4,020	4,440	+720	+1140	6
3,500	4,040	4,500	+540	+1000	9
3,700	4,280	4,740	+580	+1040	16
3,900	4,210	4,640	+310	+740	15
4,100	4,480	4,960	+380	+860	31
4,300	4,400	4,900	+100	+600	25
4,500	4,800	5,320	+300	+820	27
4,700	4,700	5,180	0	+1180	17
4,900	5,260	5,520	+360	+620	31
5,100	5,120	5,720	+20	+620	18
5,300	5,220	5,800	—80	+500	12
5,500	5,300	5,920	—200	+420	15
5,700	6,000	6,660	+300	+960	4
5,900	4,940	5,460	—960	—440	4
6,300	5,300	5,860	—1000	—440	4
6,500	5,700	6,400	—800	—100	3
6,700	6,500	7,100	—200	+400	1
Weighted average without regard to sign...			322	711	
Weighted average with regard to sign......			206	680	

Table I shows the comparison between the actual compressive strengths and the average compressive strengths predicted by the University of Maine equation and Mr. Slater's equation. The actual compressive results are given in the first column; the predicted 28-day strengths determined by the University of Maine equation in the second column; the predicted 28-day strengths determined by Mr. Slater's equation in the third column. The measure of the accuracy of prediction is the difference between the actual results and the predicted results. This difference for the University of Maine equation is given in column 4, that for Mr. Slater's equation in column 5. Column 6 shows the number of tests in each group.

If columns 4 and 5 be compared it is evident that in general the University of Maine equation gives a much better prediction of the actual results. Mr. Slater's equation overpredicts. The average weighted difference without regard to sign for the University of Maine's equation is 322 lb. against 711 lb. for Mr. Slater's equation, or the University of Maine's equation is more than twice as close to the actual results. The average weighted difference with regard to sign is 206 lb. for the University of Maine data compared with 680 lb. for Mr. Slater's data.

The conclusion could therefore be drawn that the linear equation presented by us is the most suitable equation for the prediction of the 28-day compressive strength from the 7-day compressive strength. But we have some doubt as to the validity of this conclusion. The preferred conclusion is that for accurate predictions each laboratory would best determine their equations by the method previously presented[1] for otherwise the same difficulties as those found in our use of Mr. Slater's equation are likely to occur.

Our equation was not derived for the prediction of 28-day concrete strength from 7-day concrete strength. However, it is interesting to see how closely it would apply as contrasted with the equation presented by Mr. Slater. G. W. Hutchinson has derived data suitable for this work.[2] These data were used by neither Mr. Slater nor the authors in the derivation of their respective equations and were chosen partly because of this fact and partly because the largest number of concrete tests at hand were represented. The data looked to be heterogeneous in that they are obviously composed of two centrally varying groups. We do not know why they should be heterogeneous other than that the data included several different cements which may possibly have differed significantly in their strength-producing properties. The data include 99 concretes made from standard Ottawa sand with a standard aggregate. If we determine from Mr. Hutchinson's 7-day test results the predicted 28-day test results as derived from the equation which we have previously presented and the equation which Mr. Slater presented we find that for Mr. Slater's equation there is a correlation between the predicted 28-day test results and the actual 28-day test

[1]Bulletin No. 10, loc. cit.

[2]See 1925 Proc. A. S. T. M. Part 2, Page 231. G. M. Hutchinson's discussion of "Significance of Mortar Tests" by Gowen and Leavitt.

results of 0.655 ± 0.039. Between our 28-day predicted results and the actual 28-day test results there is a correlation of 0.602 ± 0.043. The results shown in Figs. 4 and 5 do not differ significantly.

A study of Figs. 4 and 5 reveals no preference in either of the equa-

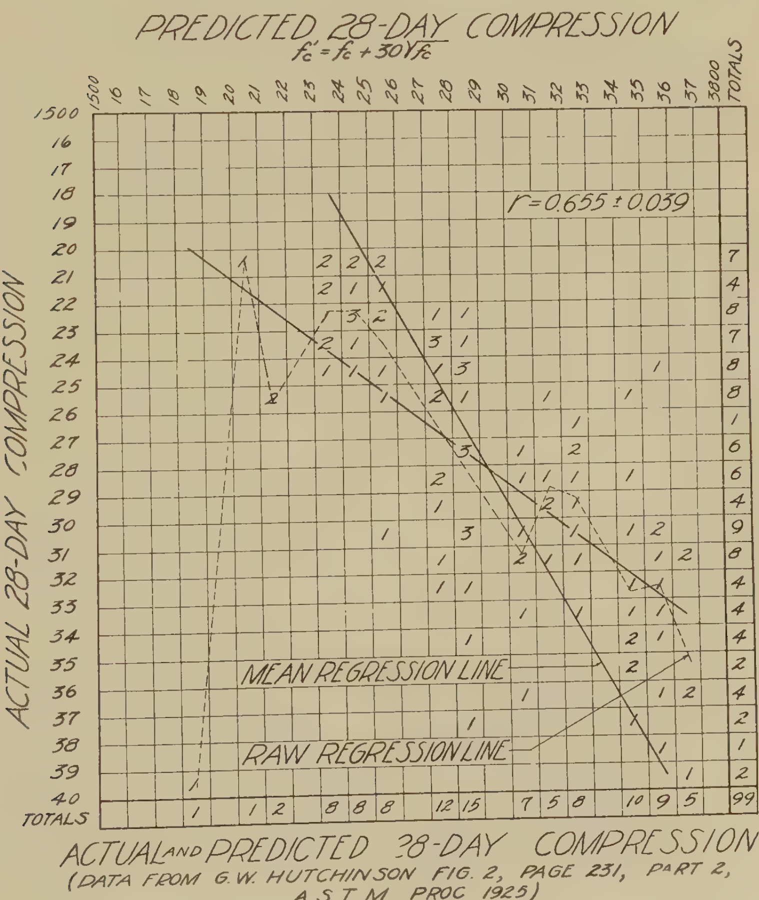

FIG. 4.—CORRELATION TABLE SHOWING THE RELATION BETWEEN MR. SLATER'S PREDICTED RESULTS AND MR. HUTCHINSON'S 99 ACTUAL 28-DAY CONCRETE TESTS.

[Gowen and Leavitt Discussion]

tions so far as these data are concerned. It is clear then that our equation is equally valid with that of Mr. Slater, even extending it to concrete. We do not, however, offer our equation for such use at this time, for we hope to be able to collect data in the future which will be more adequate for such work.

Mr. Slater shows quite distinctly that his equations for compressive strengths do not apply to tensile strength. We have been able to show elsewhere that linear equations will also apply to tensile strength data,

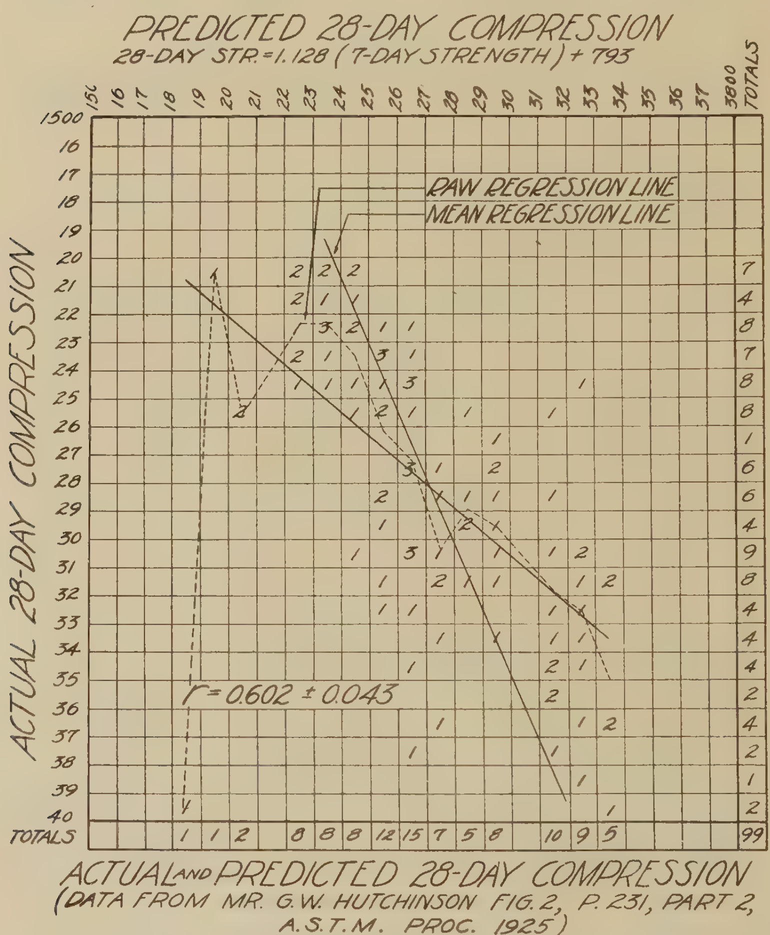

FIG. 5.—CORRELATION TABLE SHOWING THE RELATION BETWEEN UNIVERSITY OF MAINE'S PREDICTED RESULTS AND MR. HUTCHINSON'S 99 ACTUAL 28-DAY CONCRETE TESTS.

[Gowen and Leavitt Discussion]

both of Ottawa sand and on ordinary commercial or native field sands[3] as found throughout our own state of Maine.

[3]Bulletin No. 10, loc. cit.

F. E. GIESECKE* (*By Letter*).—Mr. Slater's paper is a valuable contribution to the literature relating to concrete construction. The formula submitted as expressing the relation of the 7-day to the 28-day compressive strength of concrete can be easily applied and has been used in our laboratory with satisfactory results. In order to determine how well the values calculated by means of the formula agree with actual results secured in our laboratory a study was made of practically all available tests performed in the laboratory during the past ten years. Our files were studied by C. P. Reming, laboratory assistant, who found records of 137 tests to which the formula might be applied. The tests were divided into four groups and each group represented in a separate figure. Mr. Giesecke.

Fig. 1 represents the results of 51 tests of 2-in. mortar cubes.

Fig. 2 represents the results of 34 tests of 2 x 4-in. mortar cylinders.

Fig. 3 represents the results of 36 tests of 6 x 12-in. concrete cylinders.

Fig. 4 represents the results of 16 tests of 3 x 6, 4 x 8, and 8 x 16-in. concrete cylinders.

The test results recorded in these figures were practically all secured by G. A. Parkinson, assistant testing engineer. Mr. Reming's calculations which form the basis of the figures were checked by H. R. Thomas, testing engineer.

A study of the diagrams shows:

A. That the test results secured with 2-in. mortar cubes, as shown in Fig. 1, are somewhat lower than those calculated by the formula;

B. That for the exceptionally strong concrete represented in Fig. 3 the actual results are higher than those calculated by the formula;

C. That for the ordinary type of concrete, having a 28-day strength of about 2,500 lb., the values calculated by the formula agree very well with the actual results secured in our laboratory.

*Director, Engineering Experiment Station, University of Texas.

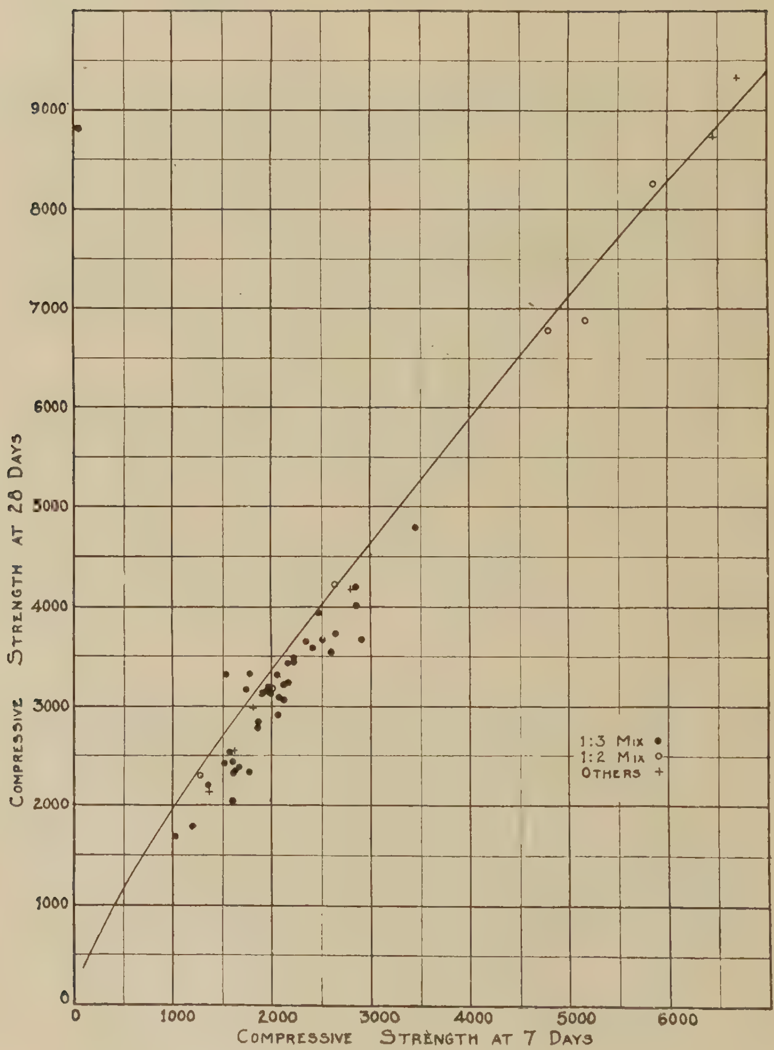

FIG. 1

[Giesecke Discussion]

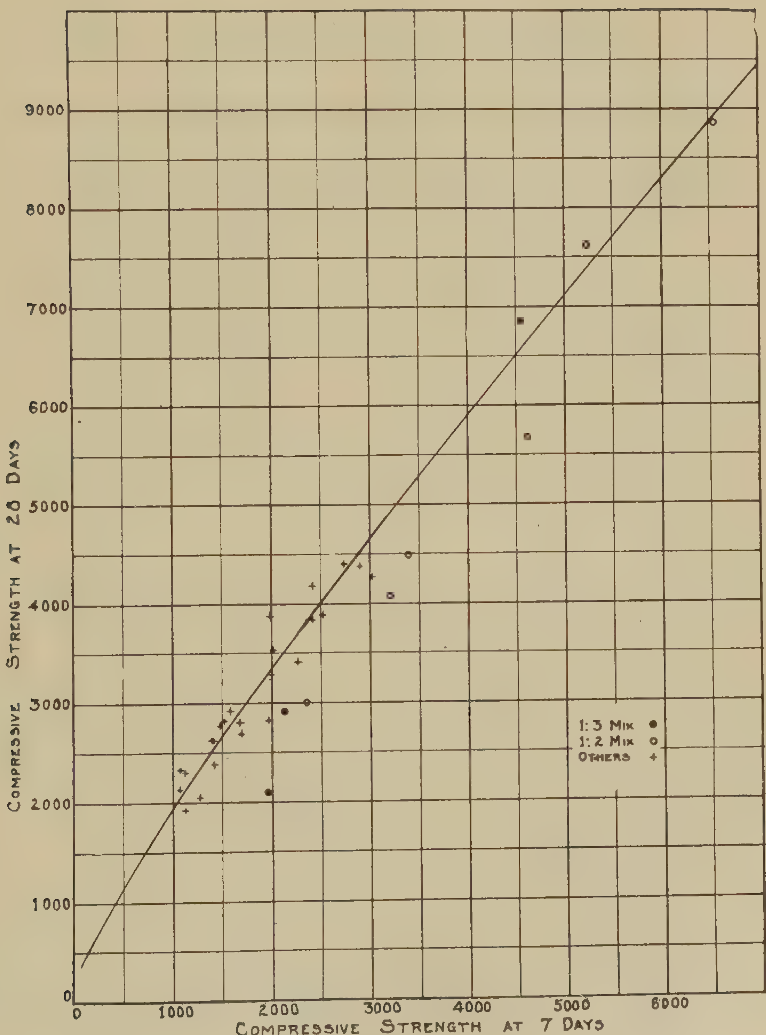

FIG. 2

[Giesecke Discussion]

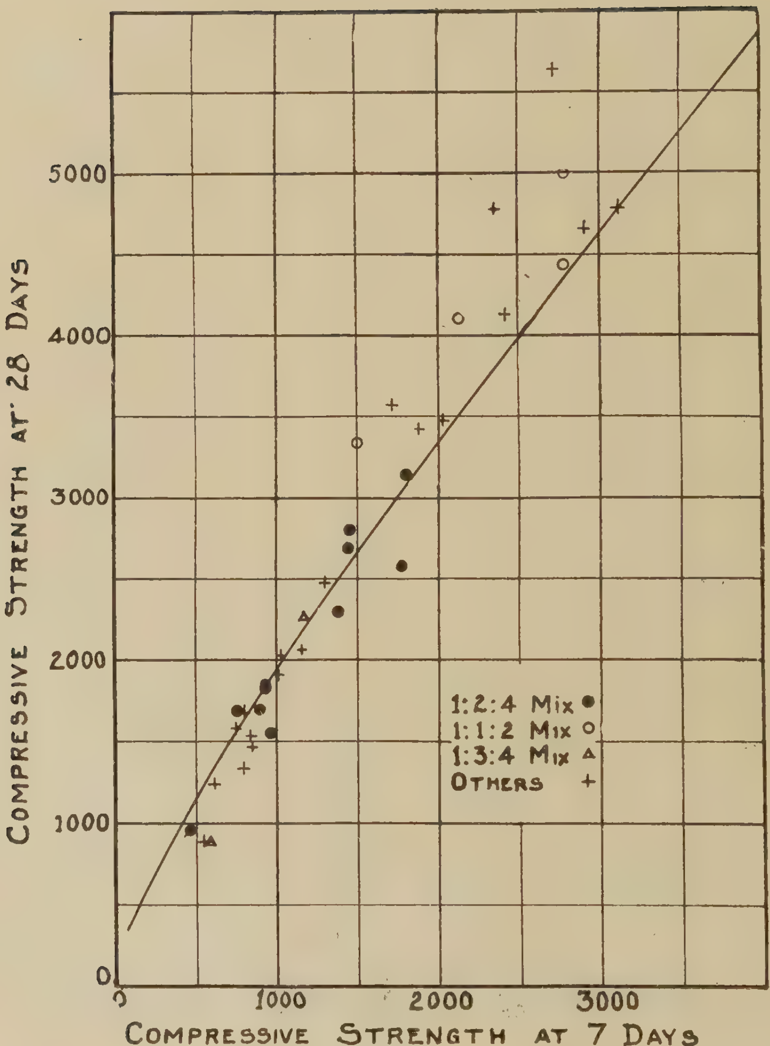

FIG. 3

[Giesecke Discussion]

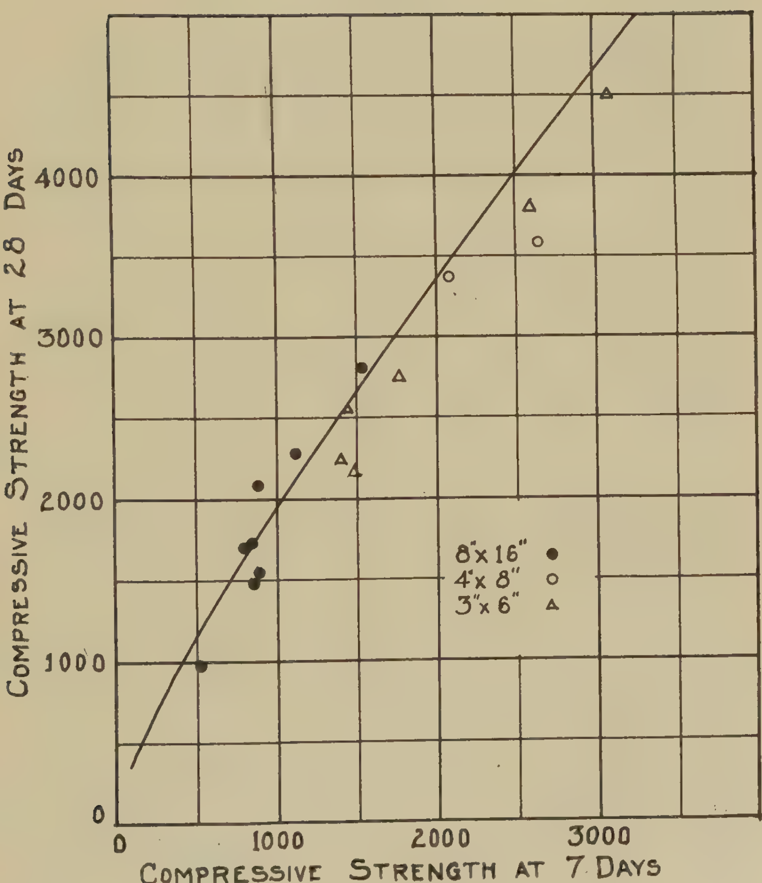

FIG. 4

[Giesecke Discussion]

Mr. Bauer.

EDWARD E. BAUER* (*By Letter*).—The writer recently made some tests at the University of Illinois which bear out the relationship between 7-day and 28-day strengths as expressed by the equation

$$f'_c = f + 30 \sqrt{f}$$

The results are given in Table I and shown graphically in Fig. 1.

Six different brands of cement were used in making concrete and mortar; crushed stone was used as a coarse aggregate and sand as a fine aggregate in three mixes and with two consistencies for each mix. Screened gravel was used as a coarse aggregate and sand as a fine aggregate in six mixes and with two consistencies for each mix. With the exception of one point, all points come fairly close to the curve expressed by the above equation.

It was felt that all cements might not gain strength at the same rate and thus affect the reliability of the relationship. In order to check this point all the brands of cement available in Champaign and Urbana (except Miami Portland cement[1]) were secured. Six 6 x 12-in. cylinders of 1 : 2 : 3½ concrete were made using each brand of cement. Six 2 x 4-in. cylinders of 1 : 2 mortar were made using each brand of cement. Three of a kind then were tested at 7 days and three at 28 days.

There was quite a wide variation in the strengths of the concrete at both 7 days and at 28 days. The 7-day strength of the concrete varied from 1,650 to 2,620 lb. per sq. in., and the mortar from 2,460 to 4,340 lb. per sq. in. The 28-day strengths of the concrete varied from 3,130 to 4,530 lb. per sq. in., and the mortar varied from 4,320 to 6,000 lb. per sq. in. The concrete and the mortar gained strength from 7 days to 28 days however according to the relationship given by Mr. Slater.

General Description of Tests.—The same cement was used in all the tests, except in the group where the cement was the variable. A coarse sand, having a fineness modulus of 3.30, was used in all the tests. Except in one set the coarse aggregate was a screened gravel having a fineness modulus of 7.50. In the other set the coarse was a broken stone from Greencastle, Indiana. It was about as coarse as the screened gravel. All consistencies were workable. Enough concrete to make six cylinders was mixed in a tilting drum mixer. Enough mortar was hand mixed to make six 2 x 4-in. cylinders. The cylinders were removed from the forms at the end of 24 hours and placed in the moist room until just before testing. The cylinders were capped with dental plaster of paris and the large cylinders tested in a 200,000-lb. hydraulic testing machine and the small cylinders in a 30,000-lb. knife-edge machine.

Henderson, Kentucky, Tests.—During the summer of 1924 the writer received some specimens from a construction job in Henderson, Ky., to be

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*Instructor in Civil Engineering, University of Illinois.

[1]Gains strength more rapidly than most portlands. It is ground so that 92 per cent passes the 200 mesh sieve.

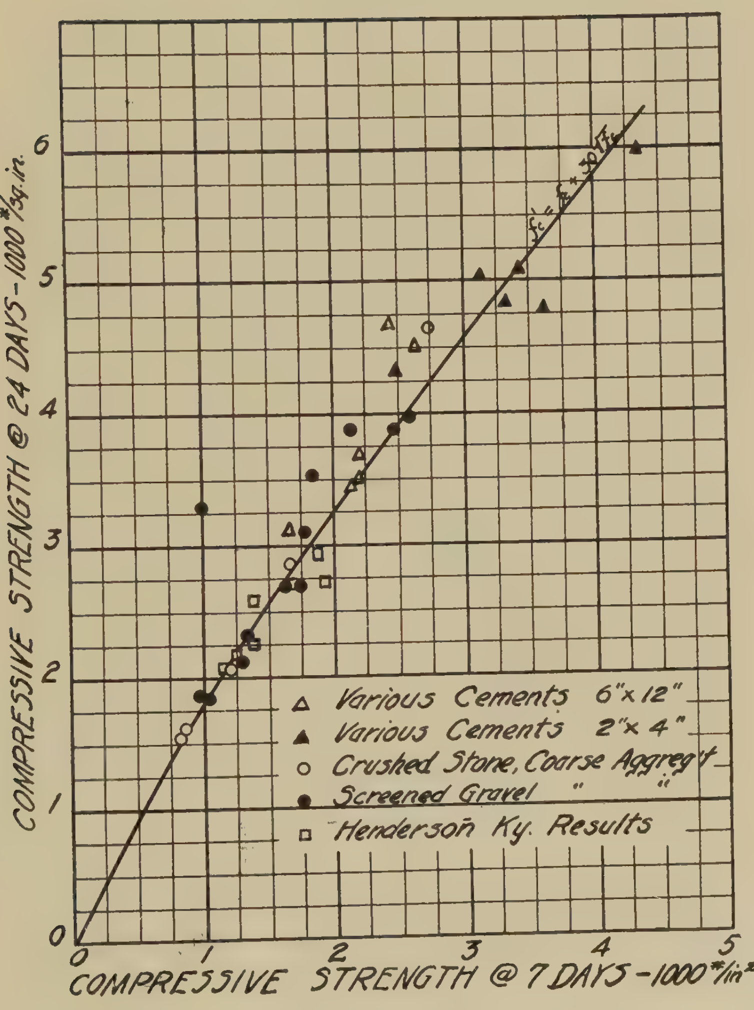

FIG. 1.—RELATION BETWEEN 7- AND 28-DAY COMPRESSIVE STRENGTH OF MORTAR AND CONCRETE.

[Bauer Discussion]

tested in compression. The results of the tests are given in Table II. The points are plotted in Fig. 1. Most of these points fall slightly under the curve.

Relation between 7-day and 28-day Tensile Strengths of Mortars.—In Fig. 2 are plotted the results of some tension tests of mortars, showing the

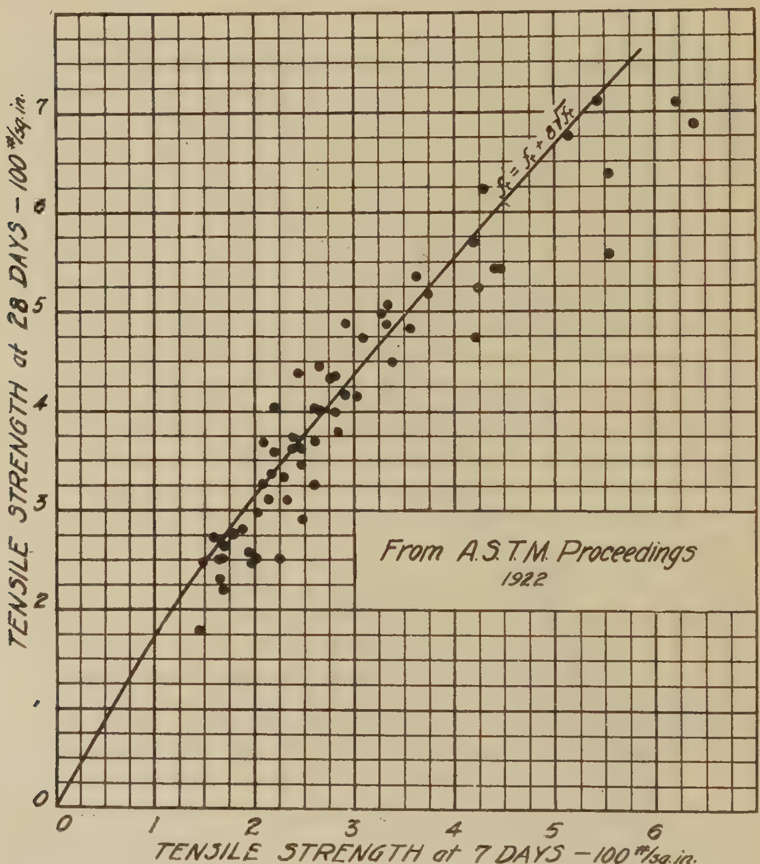

FIG. 2.—RELATION BETWEEN 7- AND 28-DAY TENSILE STRENGTHS OF MORTARS.
[Bauer Discussion]

relationship between 7-day and 28-day strengths.[2] The results fall along a curve represented by the equation

$$f'_t = f_t + 8\sqrt{f_t}$$

The results given by Mr. Slater in Fig. 15 agree fairly well with the curve given above.

TABLE I.—RESULTS OF TESTS TO SHOW RELATIONSHIP BETWEEN 7 AND 28-DAY COMPRESSIVE STRENGTHS OF CONCRETE.

CONCRETE. Various Cements. 1:2:3½ by weight. 6 x 12 in. Cylinders.			MORTAR. Various Cements. 1:2 by weight. 2 x 4 in. Cylinders.			CONCRETE. Crushed Stone Coarse Aggregate. Various Mixes and Consistencies. 6 x 12 in. Cylinders.			CONCRETE. Gravel Coarse Aggregate. Various Mixes and Consistencies. 6 x 12 in. Cylinders.			CONCRETE. Gravel Coarse Aggregate. Various Mixes and Consistencies. Student Tests. 6 x 12 in. Cylinders.		
Cement	7 day	28 day	Cement	7 day	28 day	Mix	7 day	28 day	Mix	7 day	28 day	Mix	7 day	28 day
A	1,650	3,130	A	2,460	4,320	1:1:2	2,740	4,640	1:1:2	2,445	3,890	1:1.8:3.3	2,130	3,885
B	2,110	3,450	B	3,330	4,850	1:1:2	1,670	2,850	1:1:2	2,590	3,980	1:1.8:3.3	1,630	2,695
C	2,180	3,510	C	3,140	5,050	1:1½:3	1,210	2,060	1:1½:3	1,830	3,510	1:3.4:3.9	990	3,310
D	2,180	3,700	D	3,605	4,800	1:1½:3	820	1,540	1:1½:3	1,035	1,830	1:2.4:3.9	1,295	2,120
E	2,620	4,530	E	4,340	6,000	1:2:3	1,670	2,700	1:2:3	1,780	3,100	1:3:4.5	1,730	2,695
F	2,420	3,680	F	3,410	5,120	1:2:3	870	1,620	1:2:3	980	1,885	1:3:4.5	1,354	2,310

TABLE II.—TESTS OF CONCRETE RECEIVED FROM HENDERSON, KY., IN 1924.

Number of Specimens	Kind of Specimen	Strengths 7 days	28 days
1	6 x 12-in. cylinder	1,150	2,080
1	6 x 12-in. cylinder	1,875	2,930
1	6-in. cube	1,391	2,264
1	6-in. cube	1,915	2,720
1	6 x 12-in. cylinder	1,270	2,190
1	6 x 12-in. cylinder	1,380	2,580

[2]Data given in paper by Frank E. Richart and Edward E. Bauer on "Relations Between Voids and Plasticity of Cement Mortars at Different Relative Water Contents," in *Proceedings* A. S. T. M., 1922, Vol. 22, pp. 385-403.

Construction of the Wilson Dam.

By M. C. Tyler.*

The Muscle Shoals of the Tennessee River, since the earliest days of our history, have been a serious obstruction to the navigation of the river. They are mentioned by George Washington in his diaries, during his first term as President, and described by him as a barrier against water transportation except downstream at high water stages; this at the time when settlers from the Carolinas were beginning to push west into the Tennessee Valley, and when movement of freight through the wilderness of Indian country except by water was practically impossible.

The Muscle Shoals are situated about 125 miles south of Nashville, Tennessee, about 100 miles north northwest of Birmingham, Alabama, and about 140 miles east of Memphis, Tennessee. Florence and Sheffield, Alabama, are located on the river at the foot of the Shoals. The distance by river from Chattanooga to Paducah is 460 miles and the fall is 343 ft. One hundred and four feet of this total occurs in the 18 miles just above Florence.

Between 1887 and 1889, a lateral canal with nine locks was built around the worst part of the Shoals, which made navigation possible by light-draft boats both up and down stream for about six months of the year.

The discharge of the Tennessee River at Florence varies between wide limits. The maximum flood of record is nearly 500,000 sec.-ft., while the lowest dry season flow is below 6,000 sec.-ft. On the average, there is an available water power of 245,000 K.W., or over, for 50 per cent of the time, and 500,000 K.W., or over, for 20 per cent of the time.

With the growth of the electric power industry in Alabama the development of the fall of the Muscle Shoals for power purposes became economically feasible, provided it could be combined with a navigation project and the latter could bear its fair share of the cost; and provided further that the hydro plant, or plants, at Muscle Shoals could be tied in for operation with existing and new steam plants and hydro plants on other water sheds to the South. In 1916, in compliance with directions of Congress a report was submitted by the Corps of Engineers of the Army recommending the construction of three dams with locks, two of which, Nos. 2 and 3, were to be power dams. Part of the cost was to be borne by the government and part by the Alabama Power Co. The power company was to install all power equipment at its own expense and to use all the power generated except the small amount required for operation of locks.

While this report was under consideration an act providing for the construction of nitrate plants for the production of fertilizer in time of peace and nitrates for explosives in time of war, became law. Soon after,

*Major, Corps of Engineers, U. S. Army.

the United States entered the World War and Muscle Shoals was selected as the location for two war nitrate plants, which were built, and one of which was in operation at the time of the armistice. The nitrate plants were located at Muscle Shoals because the raw materials, coal and coke, could be obtained nearby and the Tennessee River could be made to furnish hydro-electric power.

In 1918, and as a war measure, the construction of Dam No. 2 was begun, and was prosecuted until June, 1921, when for lack of funds work was suspended for over a year. In 1919 the Secretary of War ordered that

FIG. 1.—LOCK, DAM AND POWER HOUSE FROM THE AIR.

the dam be named "Wilson Dam." Since 1922 the work has been actively pushed and except for minor items is now completed.

The construction work has been done by hired labor and government-owned plant, administration of the work being in accordance with the laws and regulations governing the Corps of Engineers of the Army. The consulting and designing engineers are Hugh L. Cooper and Co., under a contract with the Chief of Engineers, U. S. Army. The consulting engineers have inspected the work for the Chief of Engineers through a resident engineer and a force of inspectors on the job. Hydraulic and electrical equipment, spillway gates, lock gates, draw bridge, etc., have been purchased and installed by contract after competitive bidding.

The site of the work is in a rock gorge about 4,300 ft. between bluffs. The foundations are limestone in horizontal ledges, with seams generally tight.

The locks are two in flight each with an available length of 300 ft., width of 60 ft. and lift of 44½ ft. The locks contain 78,000 cu. yd. of concrete.

The spillway dam has its crest 79 ft. above the original bed of the river. Fifty-eight spillway gates 38 ft. x 18 ft. maintain the lake level

FIG. 2.—AERIAL VIEW OF WILSON DAM POWER HOUSE.

18 ft. above the spillway crest. A concrete arch bridge crosses the dam and forebay structure and serves as a supporting platform for gate operating machinery and as a main trunk highway with a 20 ft. pavement. Seven hundred and thirteen thousand cubic yards of concrete were placed in the dam section.

The forebay structure is in line with the dam. The powerhouse, 1,200 ft. long, contains wheel pits for eighteen main units or a total capacity when all units are installed of 610,000 H.P. The present installation is eight main units, four of 30,000 H.P. and four of 35,000 H.P. The powerhouse section contains 524,000 cu. yd. of concrete.

The switch house and oil circuit breaker buildings are situated at the

south end of the powerhouse on top of the bluff and house the switch boards and switching equipment.

All masonry is massive in design. Steel reinforcement was used only in penstocks, draft tubes, powerhouse superstructure, switch house and oil circuit breaker buildings.

Five bags of cement per cubic yard were used in all mass work except where reinforced and except in bridge piers, arches and parapets. Six bags

FIG. 3.—CONSTRUCTION OF THE LOWER LOCK.

of cement per cubic yard were used in penstocks, scroll cases, draft tubes, buildings and bridge piers, arches and parapets.

The total yardage of concrete is......	1,316,000 cu. yd.
Earth exacavation	734,000 " "
Rock excavation	690,000 " "

Twenty-five pounds of hydrated lime per cu. yd. were used in all concrete.

Transportation.—Rail transportation was within easy reach on both sides of the river. The work was handled in two divisions. One responsible for the locks and dam based on the north side of the river with rail connection to the Southern and L. & N. railways in East Florence, about two miles from the dam; the other responsible for the powerhouse section,

FIG. 4.—EXCAVATION FOR HEEL TRENCH OF DAM.

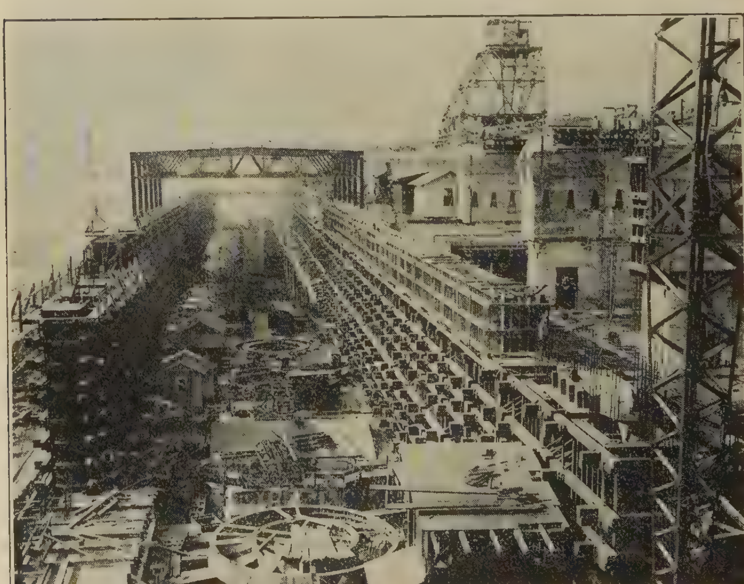

FIG. 5.—POWER HOUSE SUPERSTRUCTURE UNDER CONSTRUCTION.

based on the south side of the river with rail connection to the Southern Ry. in Sheffield, about four miles from the work. Each division had its own shops, air-compressor plants, railway yards, storage and warehouses and construction equipment. Twenty-nine miles of standard-gage railroad in connections to main lines, to the sand and gravel dock and in yards served the work. Standard-gage equipment was used throughout.

Electric power at a low rate was available from the lines of the Alabama Power Co. within a mile and a half of the south end of the dam. Power was delivered to two substations on the work at 13,000 volts, one on the south bank of the river and one on Jackson Island. From the Jackson Island substation a 13,000-volt line was carried four miles to the sand and gravel dock. Two thousand three hundred-volt feeders were led from the two substations to all parts of the work. Where voltage less than 2,300 was required, the 2,300-volt lines were carried as near the point of use as possible and there stepped down to the proper voltage.

Generally speaking, all mixing plants and air compressors were operated at 2,300 volts; all construction cranes at 220 volts and lighting at 110 volts. The main items of plant electrically operated were:

7	Air compressors	325 K.W. each
3	McMyler cranes	150 " "
7	Terry & Co. cranes	100 " "
1	Mixing plant, 2 4-cu. yd. mixers	275 "
3	Mixing plants, 2 2-cu. yd. mixers	125 " "
2	10-ton guy derricks	90 " "
2	Gantry cranes	250 " "
1	Rock-crushing plant	125 " "
	Pumps	200 "

The maximum demand for any one month during the progress of the work was 2,700 K.W. and the power consumed ranged between three-fourths and one and one-fourth million kilowatt hours per month.

All parts of the work were lighted by temporary electric lines. Flood lights were installed at points about the cranes and bluffs and were semi-permanent. Others were moved from day to day as the work progressed. A stock of lead-covered, steel-armored cable of various capacities, was kept on hand. This was especially valuable where overhead lines would have interfered with cranes.

Band saws, rip saws, drill presses and small pumps, all with individual motors, were used about the work wherever needed. They could be moved at one lift by the construction cranes. During the entire period of the work not one fatal accident occurred from contact with electric power or light lines.

All electrical equipment proved highly reliable and time lost on account of failures of power lines or electrical apparatus was negligible.

Construction Bridge.—The dam was built from a construction bridge of concrete piers and steel girders carrying railroad tracks the full length of the dam. The bridge trackage connected with tracks on the south side

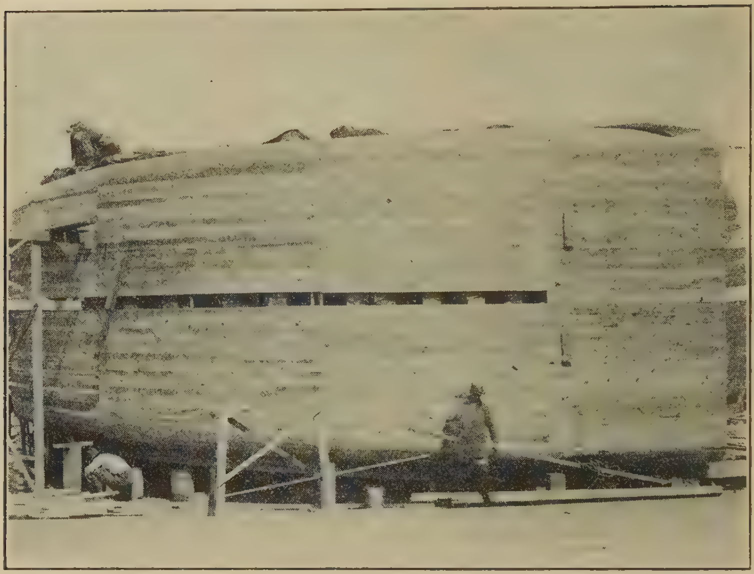

FIG. 6.—SCROLL-CASE FORM.

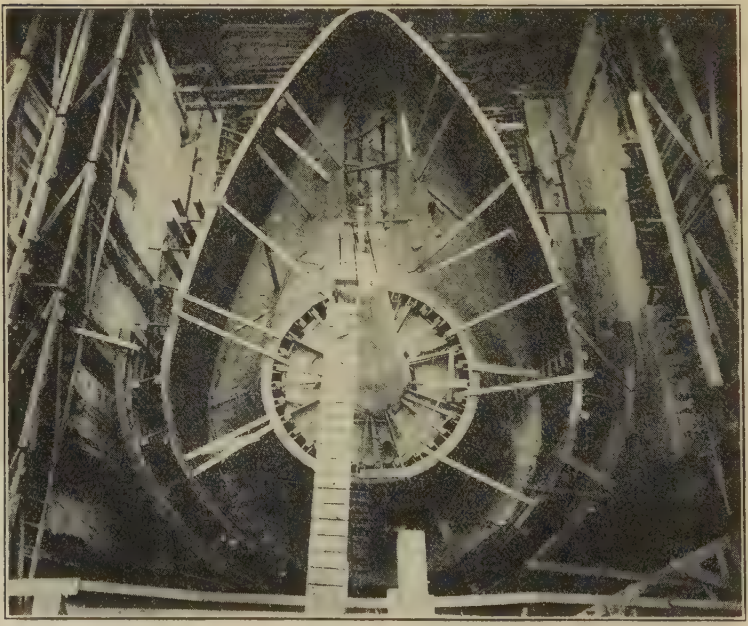

FIG. 7.—DRAFT-TUBE FORMS.

of the river over the downstream powerhouse cofferdam, so that for the greater period of the work rail connection between the two sides of the river was maintained.

Sand and Gravel.—Coarse and fine aggregates were secured from the river by dredging on bars located from nine to eleven miles below Florence and twelve to fourteen miles below the work. Two cutter head, 15-in. pipe line dredges were used. The dredges discharged into the screens of a screening and washing barge lashed at the stern of the dredge. From the screen barge sand and gravel discharged by gravity onto deck barges.

Barges were towed by stern-wheel river towboats from the dredges to the unloading dock at Florence. Here transfer was made to rail, since the river was too shoal to permit of reaching the work by boat. Two electrically-operated gantry cranes with 4-cu. yd. clam shells placed sand and gravel in bins from which 20-cu. yd. air-dump cars were loaded. From the dock sand and gravel was hauled direct to the various mixer plants and dumped, or to dump storage on Jackson Island. In low water the upstream towing was difficult, as only a 4½-ft. depth was available and currents were as swift as five miles per hour. Stockpiles were kept well built up during the high water season to avoid a shortage during the low water season when navigation was difficult.

Four towboats and about twenty deck barges varying in capacity from 200 to 400 cu. yd. were kept in service. The rate of delivery to the dock was about 2,000 cu. yd. per day.

All concrete was transported in buckets carried on flat cars from the mixer to the placing cranes. Four-cubic yard and 2-cu. yd. buckets were used, two 4-cu. yd. or four 2-cu. yd. buckets to a car. Concrete trains were hauled by 25-ton and 40-ton saddle-tank locomotives.

Electric Cranes.—Seven electric cranes furnished by Terry and Co. operating from the construction bridge and on the downstream side of the dam placed the 700,000 cu. yd. in the dam. These cranes had the following characteristics:

Capacity—10 tons at 75-ft. radius.
Travel speed—100 ft. per minute.
Hoisting speed—66 ft. per minute at full load.
Hoist motor—75 H.P., 220 amp., 575 R.P.M.
Travel motor—52 H.P., 152 amp., 570 R.P.M.
Swinging motor—22 H.P., 65 amp., 850 R.P.M.
General Electric induction motors, variable speed, Lidgerwood hoists.

In order to reach above the arch bridge and place concrete therein, a hammerhead was added to the booms and the load cut down from 4 cu. yd. to two. Also in some cases 100-ft. booms were substituted and 2-cu. yd. buckets handled.

Three McMyler cranes covered the powerhouse, two to forebay structure working on the upstream side and one to the draft tube and power-

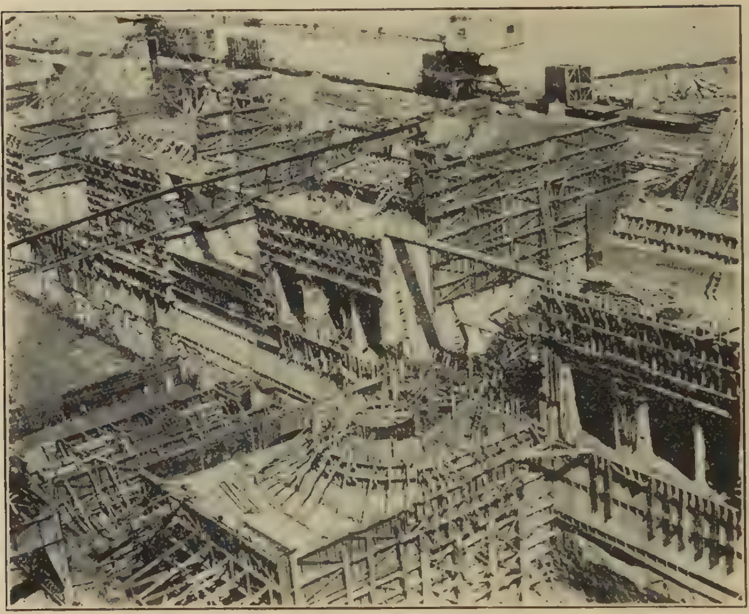

FIG. 8.—FOREBAY AND SCROLL-CASE FORMS.

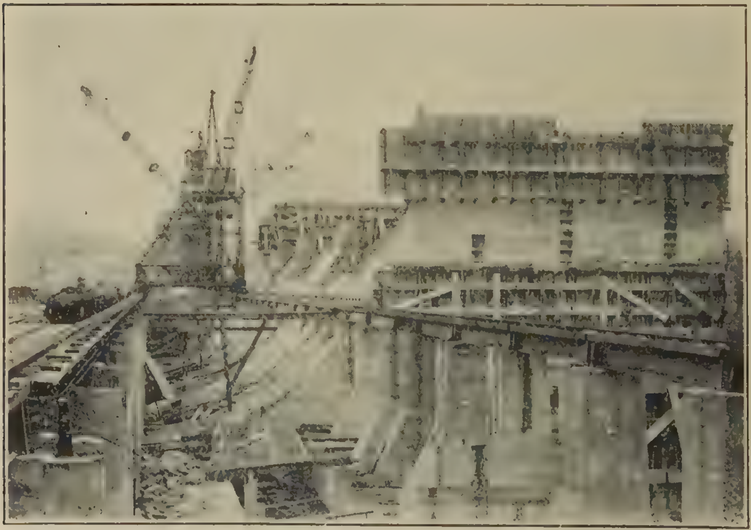

FIG. 9.—CONSTRUCTION BRIDGE WITH TRAVELER-MOUNTED CRANES.

house superstructure working from downstream. These cranes had the following characteristics:

Capacity at 100-ft. radius	10 tons		
Capacity at 50-ft. radius	20 "		
220-volt Westinghouse induction motors, variable speed			
Hoisting	100 H.P.,	278 amp.,	690 R.P.M.
Travel	75 "	195 "	870 "
Swinging	25 "	70 "	1150 "

Travel speed—150 ft., hoisting speed at full load 100 ft. per minute.

As soon as the forebay structure was up full height and a Terry crane could be spared from the dam, it was placed on top of the work, its track gage having been narrowed. Another Terry from the dam was transferred to the downstream side of the powerhouse structure.

On the completion of the forebay structure one of the McMyler cranes was taken down and re-erected in the lower lock. Another was taken down and re-erected on top of the bluff to cover the switch house and oil circuit breaker building and later the third McMyler was similarly placed. One hundred and fifty-foot booms were built on the work and substituted for the 115-ft. booms. With the long booms the cranes handled 2-cu. yd. buckets without any trouble.

Mixing Plants.—Four main mixing plants were used.

(a) One containing two 2-cu. yd. Lakewood mixers located upstream of the forebay structure. This mixing plant was at a low level and its work had to be completed before closure of the dam was very far along.

Coarse and fine aggregate was measured in measuring boxes before discharging into mixer hoppers. Water was measured in measuring tanks.

	Cubic Yards
The maximum daily output of this plant was	1,432
The maximum monthly output of this plant was	31,181
Total output	289,312

(b) One containing two 2-cu. yd. Smith mixers located downstream of the tailrace excavation and within the powerhouse cofferdam.

	Cubic Yards
The maximum daily output of this plant was	1,248
The maximum monthly output of this plant was	18,574
Total output	116,522

(c) Jackson Island plant containing two 4-cu. yd. Smith mixers.

In this plant cement was dumped from sacks onto a belt conveyor and transferred to a bin in the mixer from which batch cars were loaded for dumping into mixer hoppers.

	Cubic Yards
The maximum daily output of this plant was	2,715
The maximum monthly output of this plant was	46,698
Total output	419,601

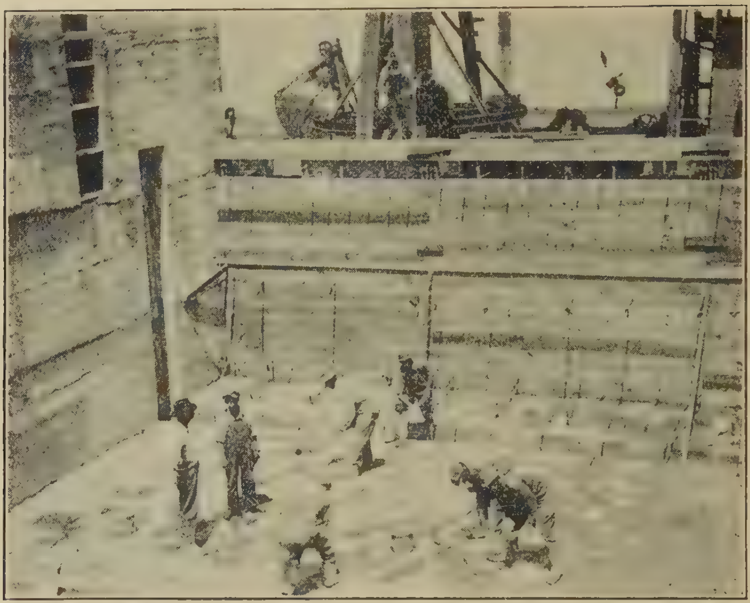

FIG. 10.—CLEANING TOP OF CONCRETE PREPARATORY TO POURING NEXT LIFT.

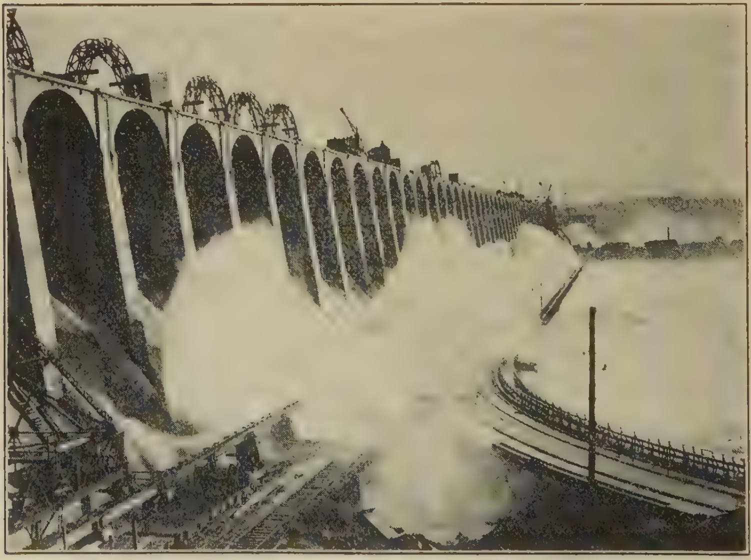

FIG. 11.—WATER POURING OVER SPILLWAY CREST.

This plant also had in connection therewith a rock-crushing plant, one 48-in. Allis-Chalmers gyratory crusher and four 14-in. secondary crushers with belt conveyors and screens.

(d) North shore mixer plant, containing two 2-cu. yd. Smith mixers.

	Cubic Yards
The maximum daily output of this plant was...	1,441
The maximum monthly output of this plant was	25,454
Total output	187,142

All main mixer plants except the first one described were equipped with bucket elevators for handling gravel and clam shell derricks for sand.

(e) The small mixer plant for the construction of the switch house contained two ¾-cu. yd. mixers operated by air.

Forms.—As previously stated, each side of the river had its own carpenter shop, including motor-driven, wood-working machinery, and lumber storage. All forms were made at the shops where possible and were transferred on car to the cranes for placing.

All forms were very substantially built in order to maintain the lines of the work and were securely braced and rodded. Scrap pipe was used in great quantities for interior bracing. The penstock, scroll case and draft-tube forms were very complicated and required a high order of skill in fabrication. Their construction more nearly resembled that of boat building or pattern making than the usual run of form construction. All these forms were built in sections or quadrants and the whole fitted together on the layout platforms. Sheeting generally was so laid as to be in the direction of travel of water in water passages. Sections of forms were handled from the loading platforms by locomotive cranes and loaded on flat cars and moved by rail to the construction cranes.

Penstock forms were removable and were used over, some as many as five times. Scroll case and draft-tube forms from the nature of the work could be used but once. Panel forms for lock and dam were used over after repair until the faces were too bad for further use.

At the height of the work (1923) about 150 carpenters and 125 helpers were employed in carpenter shops, and 400 carpenters and 175 helpers in erecting forms on the work.

Placing Concrete.—All loose rock in excavation was removed and rock bottom thoroughly gone over with picks, air and water to remove any remainder of mud seams. Neat cement was spread over the rock and wet down before concrete was placed. Pours were generally 5 ft. in depth. Concrete was dumped from buckets and shoveled and booted into horizontal layers. Particular care was used to get good faces on all pours whether to be exposed or against other work.

The top of each pour was broomed with wire brooms while still green. Before the next pour was placed, it was picked and blown with air and water to remove all laitance.

Concrete was placed on three shifts in the powerhouse division. During the period of closing the dam it was found advantageous to use the

construction cranes on the first shift for placing closure gates and stop logs and forms and to limit concreting to the second and third shifts. The maximum number of concrete placing gangs employed was twenty-six, consisting of 380 men, or an average of 14 or 15 men per gang.

The maximum amount of concrete placed in any one month was 71,900 cu. yd. in October, 1923. Concrete placing records by years were as follows:

	Cubic Yards.		Cubic Yards.
1920	110,903	1923	409,788
1921	152,010	1924	504,645
1922	72,445	1925	65,758

Shut down from June, 1921, to September, 1922

Cost of Concrete.—Costs of concrete in the various parts of the work were as follows:

UNIT COST TO DEC. 31, 1925.

Description	Field	Overhead	Total (Inc. Overhead)
Lock Concrete:			
Cement	$3.53	$1.64	$5.17
Sand	0.54	0.25	0.79
Gravel	1.05	0.49	1.54
Crushed Rock	0.08	0.04	0.12
Plumstone	0.02	0.01	0.03
Mixing	1.05	0.49	1.54
Hauling	0.70	0.33	1.03
Placing	0.87	0.41	1.28
Form Building	0.63	0.29	0.92
Form Erection	2.02	0.95	2.97
Form Removal	0.29	0.14	0.43
Total per Cu. Yd.	$10.78	$5.04	$15.82
Dam Concrete:			
Cement	$3.22	$1.50	$4.72
Sand	0.59	0.28	0.87
Gravel	1.05	0.49	1.54
Crushed Rock	0.17	0.08	0.25
Plumstone	0.00	0.00	0.00
Mixing	0.80	0.37	1.17
Hauling	0.36	0.17	0.53
Placing	0.70	0.33	1.03
Form Building	0.61	0.29	0.90
Form Erection	1.28	0.61	1.89
Form Removal	0.22	0.10	0.32
Total per Cu. Yd.	$9.00	$4.22	$13.22

Description	Field	Overhead	Total (Inc. Overhead)
Bridge Concrete (Arches):			
Cement	$3.58	$1.68	$5.26
Sand	0.46	0.22	0.68
Gravel	1.09	0.51	1.60
Mixing	1.06	0.50	1.56
Hauling	0.63	0.29	0.92
Placing	1.52	0.71	2.23
Form Building	2.65	1.24	3.89
Form Erection	5.16	2.41	7.57
Form Removal	1.31	0.61	1.92
Total per Cu. Yd..	$17.46	$8.17	$25.63
Mass Concrete, Powerhouse:			
Cement			
Sand			
Gravel			
Crushed Rock			
....	$4.90	$2.29	$7.19
Plumstone	0.01	0.00	0.01
Mixing	0.72	0.34	1.06
Hauling	0.43	0.21	0.64
Placing	0.82	0.38	1.20
Form Building	0.83	0.39	1.22
Form Erection	1.37	0.64	2.01
Form Removal	0.35	0.16	0.51
Total per Cu. Yd..	$9.43	$4.41	$13.84
Mass Reinforced, Powerhouse:			
Cement	$3.70	$1.73	$5.43
Sand	0.58	0.27	0.85
Gravel	1.14	0.53	1.67
Crushed Rock	0.10	0.05	0.15
Plumstone	0.00	0.00	0.00
Mixing	1.04	0.49	1.53
Hauling	0.67	0.31	0.98
Placing	1.30	0.61	1.91
Form Building	3.30	1.54	4.84
Form Erection	2.43	1.14	3.57
Form Removal	0.62	0.29	0.91
Total per Cu. Yd..	$14.88	$6.96	$21.84

Description	Field	Overhead	Total (Inc. Overhead)
Reinforced Powerhouse Section:			
Cement	$3.69	$1.73	$5.42
Sand	0.52	0.24	0.76
Gravel	1.25	0.58	1.83
Crushed Rock	0.00	0.00	0.00
Plumstone	0.00	0.00	0.00
Mixing	2.21	1.03	3.24
Hauling	1.47	0.69	2.16
Placing	3.48	1.63	5.11
Form Building	7.33	3.43	10.76
Form Erection	8.19	3.84	12.03
Form Removal	1.86	0.87	2.73
Total per Cu. Yd..	$30.00	$14.04	$44.04
Switch House:			
Cement	$3.64	$1.70	$5.34
Sand	0.54	0.25	0.79
Gravel	1.28	0.60	1.88
Crushed Rock	0.00	0.00	0.00
Plumstone	0.00	0.00	0.00
Mixing	1.62	0.76	2.38
Hauling	1.44	0.67	2.11
Placing	6.94	3.25	10.19
Form Building	22.54	10.55	33.09
Form Erection	10.95	5.13	16.08
Form Removal	2.18	1.02	3.20
Total per Cu. Yd..	$51.13	$23.93	$75.06
Powerhouse Superstructure:			
Cement	$3.62	$1.69	$5.31
Sand	0.44	0.21	0.65
Gravel	1.20	0.56	0.76
Crushed Rock	0.00	0.00	0.00
Plumstone	0.00	0.00	0.00
Mixing	1.69	0.79	2.48
Hauling	1.67	0.78	2.45
Placing	5.87	2.75	8.62
Form Building	15.38	7.20	22.58
Form Erection	10.13	4.74	14.87
Form Removal	2.06	0.96	3.02
Total per Cu. Yd..	$42.06	$19.68	$61.74

NOTE: The unit prices given above are subject to slight revision downward depending upon the amount received from sale of plant.

The total cost of the work will be slightly under $46,000,000 for the eight-unit installation, exclusive of step-up transformer stations, for which an appropriation of $2,000,000 is pending in Congress. The estimated value of the navigation improvement is $9,000,000, so that the completed eight-unit installation of 230,000 K.V.A. will involve an investment of $39,000,000.

Control of Mixture and Testing of Wilson Dam Concrete.

By John W. Hall.*

The building of Wilson Dam involved the placing of approximately 1,350,000 cu. yd. of concrete. This concrete was mixed at central mixing plants, dumped into concrete buckets which were on flat cars, and conveyed by railroad to the various parts of the work, where it was deposited in the forms by cranes or derricks. The buckets used for depositing the concrete were of the bottom dump type and were, for the most part, either of 2- or 4-cu. yd. capacity. The inspection of all concrete work was performed by representatives of Col. Hugh L. Cooper, consulting engineer of this work, who had charge of the design and supervision of installation of all work on this project, including concrete and installation of all equipment.

Specifications.

That portion of the specifications prepared by Colonel Cooper which particularly relates to the concrete work is as follows:

1. Each cubic yard of concrete shall contain five bags of standard portland cement, with the following exceptions: concrete to be placed in the power house superstructure, all reinforced-concrete work, arches in the main dam, and piers in the main dam above El. 483, which concrete shall contain six bags of cement per cubic yard.

2. *Hydrated Lime.*—Each cubic yard of concrete shall contain 25 lb. of hydrated lime. If supply of lime is not available, one-half bag of cement shall be added, per cubic yard of concrete, to the cement already being used.

3. *Aggregate.*—The proportions of sand and broken stone or gravel will be determined by the resident engineer by a cube measurement of the voids in the broken stone or gravel. The quantity of sand to be mixed with the five bags of cement will be such quantity as will yield for each cubic yard of concrete, 15 per cent more mortar than will be required to fill the voids in the broken stone or gravel. Twenty-five cubic feet of stone shall be used per cubic yard of concrete. Measurements of sand and broken stone or gravel will be by volume, and when said volumes are determined, their uniform use will be assured by proper measuring boxes or spouts in the mixer, so designed and operated as to guarantee correct quantities to each batch. If gravel aggregate is used, the sand in the same will be screened out and the gravel remaining will be measured and treated as broken stone.

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*Resident Engineer, Hugh L. Cooper & Co., Florence, Ala.

4. *Sand.*—The sand used shall be free from vegetable matter, lumpy clay, or other foreign matter injurious to the permanent mortar. The sand may vary widely as to fineness and may contain not to exceed 7½ per cent of well distributed clay.

5. *Broken Stone.*—Broken stone will be crushed to sizes that will pass a 3½-in. diameter ring. Crushers will be equipped with a revolving or other form of screen system that will separate entirely the crusher dust from the broken stone, and convey the dust to a separate bin from where it will be wasted.

FIG. 1. TYPE OF DUMP-BOTTOM BUCKETS USED IN DEPOSITING CONCRETE.

6. *Mixing.*—Mixer drums will revolve a minimum of 12 revolutions or until the mortar is of a uniform color throughout. Minimum time of mixing concrete, 2 min. for the 2-yd. mixers, and 2½ min. for the 4-yd. mixers, fine sand may require a longer period.

7. *Quantity of Water.*—The quantity of water for each batch will be that minimum quantity required by the standard quaking test for freshly-mixed concrete. On this work, this quaking test will require that men dumping concrete buckets do not track deeper than 10 in. nor less than 2 in. in freshly-mixed concrete.

8. *Placing (A).*—Concrete will be dumped as soon after mixing as possible and no concrete will be placed after initial set has occurred. Simi-

larly, no concrete will be placed on concrete that has not set sufficiently to bear the new concrete without deformation of any kind.

9. *Placing (B)*.—Thickness of courses will be a minimum of 4 ft. wherever possible, and as much greater than 4 ft. as economy of forms will permit. Maximum depth of concrete courses in general 6 ft., unless specified by the resident engineer.

10. *Temperature*.—When ambient temperatures are above 90 deg. F., all green concrete will be protected with tarpaulins during the times of excess heat, and be wetted for 48 hours after placing.

11. *Rate of Deposit*.—The rate of deposit of concrete shall in no case exceed the capacity of the concrete crews to distribute the concrete properly and to spade and fork the surfaces thoroughly.

12. *Expansion Joints*.—Vertical expansion joints between apron sections and main dam and between all longitudinal sections of main dam and power house and lock walls, will occur as shown on the plans. These expansion joints will be effected by the use of 36-in. wide standard building tar paper. When the temperature is below 40 deg. F., three thicknesses of tar paper will be placed on the periphery of the permanent faces to be concreted against. When the ambient temperature is below 40 deg. F. and 80 deg., use two thicknesses, and when above 80 deg., use one thickness of tar paper. Gray felt will be used in place of tar paper in expansion joints above water level.

OPERATIONS.

Concrete Aggregate.—At least 95 per cent of the sand and gravel used for concrete work was obtained by dredging in the river about nine miles below the site, at Buck Island. The gravel and sand thus obtained were separated by revolving screens, and transported to the work separately. A certain amount of coarse aggregate was obtained by crushing the limestone rock excavated in preparation of foundations.

One department of the Hugh L. Cooper & Co. inspection force consisted of a thoroughly-equipped testing laboratory, wherein all ingredients entering into the concrete were properly tested. The gravel and sand were tested for clay or silt, and organic matter. The sand was also tested for tensile strength. In addition to this, a sieve analysis of the sand and gravel was made from time to time, particularly whenever the dredge moved to a new location. Table I shows the sieve analysis of the Buck Island sand, and Table II shows the sieve analysis of the Buck Island gravel. The moisture in the sand at the mixing plant varied from 4.1 per cent to 6.7 per cent by weight. The weight per cubic foot of sand averaged 106.9 lb. The moisture in the gravel at the mixing plant varied from 4.7 per cent to 6.6 per cent by weight. The weight per cubic foot of gravel varied from 90 to 97 lb. Some fine aggregate of size not to exceed 1 in. was needed in reinforced work, and this was obtained by a special screen at the dredging plant.

The cement used in this project was tested twice: First at the mill by a representative of the Bureau of Standards or a private testing laboratory, and second, in the Hugh L. Cooper & Co. laboratory at the job. In testing the cement at the latter laboratory, Buck Island sand was used in making 1 : 3 briquettes and comparisons were made from time to time with 1 : 3 briquettes made from the same cement using Ottawa sand. This gave a comparison of the relative merits of the Buck Island sand with standard

TABLE I.—SIEVE ANALYSES OF BUCK ISLAND SAND.

Sample No.	Date of Sample	Amount Coarser Than Each Sieve (per cent by weight)						Fineness Modulus
		100-mesh	48-mesh	28-mesh	14-mesh	8-mesh	4-mesh	
Z-181	7-25-24	97.7	85.1	43.5	26.9	16.3	5.7	275.2
Z-190	8-15-24	99.7	88.7	50.0	34.5	24.0	8.5	305.4
Z-225	9-12-24	98.5	88.3	44.7	29.7	20.1	8.1	289.4
Z-235	9-20-24	98.4	89.9	56.6	38.4	25.5	7.8	316.6
Z-250	11-20-24	98.2	85.8	44.7	30.3	10.2	7.8	277.0
Z-260	12- 9-24	97.8	85.0	46.3	31.8	22.0	9.8	292.7
Z-270	12-22-24	99.0	83.0	36.5	21.8	14.0	4.5	258.8
Z-277	1- 3-25	98.9	87.4	40.4	24.3	15.8	6.0	272.8
Z-294	1-22-25	99.3	89.5	47.1	31.6	23.1	12.5	303.1
Z-308	2- 2-25	98.8	94.1	63.0	42.1	28.0	14.5	340.5
Z-317	2- 7-25	99.2	89.7	45.5	30.5	21.4	9.0	294.3
Z-322	2-12-25	99.1	87.5	42.5	28.0	19.3	7.3	283.7
Z-351	3-10-25	97.8	77.1	33.8	18.8	12.2	5.4	245.1

TABLE II.—SIEVE ANALYSES OF BUCK ISLAND GRAVEL.

Date of Sample	Amount Coarser Than Each Sieve (per cent by weight)									Fineness Modulus
	100-mesh	48-mesh	28-mesh	14-mesh	8-mesh	4-mesh	⅜-in.	¾-in.	1½-in.	
3-19-24	...	...	...	...	97.4	96.3	82.3	54.8	35.8	....
6-12-24	...	...	...	...	....	98.0	87.3	70.3	18.0	....
6-21-24	...	...	...	...	....	94.3	76.2	68.1	6.0	....
*6-21-24	...	...	...	...	91.1	81.5	51.0	13.4	0.0	....
*6-27-24	...	...	...	...	99.6	84.6	54.5	25.4	0.0	....
8-12-24	...	...	...	...	....	65.2	58.5	42.8	9.4	....
*1-24-25	100	100	100	100	100	99.8	87.9	24.5	0.0	712.2
1-31-25	100	100	100	100	100	99.6	99.6	95.1	46.3	840.6
2- 4-25	100	100	100	100	100	99.8	98.3	48.7	4.2	751.0
2- 7-25	100	100	100	100	100	99.6	99.3	89.9	28.3	817.1

* Specially graded for use in reinforced concrete where maximum size gravel could not exceed 1 in.

Ottawa sand. Also, by using Buck Island sand for all tests in the laboratory, a comparison was made with the mortar briquettes made from the concrete deposited in the work. Our tests showed that the Buck Island sand tested at 380 lb. in 7 days, as compared with 301 lb. for the Ottawa sand, and in 28 days the Buck Island sand tested at 485 lb., as compared with 377 lb. for Ottawa sand. In other words, the Buck Island sand tested 26 per cent stronger than the standard Ottawa sand in 7 days, and

29 per cent stronger in 28 days. The cement was stored in bags in warehouses near the mixing plants, and this cement was used before it became four months old. In any case where cement had stood in the warehouse for a period of more than four months, this cement was re-tested and used only provided it proved satisfactory.

Mixing.—The mixing of the concrete was accomplished in the usual manner, using the volumetric method of measuring the sand and gravel. Both sand and gravel were measured in hoppers which were properly graduated, to provide the number of cubic feet of sand or gravel desired. Five or six bags of cement were added per cubic yard, depending upon where the concrete was to be deposited. The control of the mixture was at all times under the direction of an inspector representing Hugh L. Cooper & Co., who designated the proper amount of sand and water to be used in the mix. This inspector made void tests at frequent intervals throughout the day, to determine the voids in the coarse aggregate, and varied the amount of sand accordingly. The sand and gravel bins were replenished by a conveyor, sometimes from the stockpile, and sometimes from dump cars immediately after arrival at the mixing plant. The quantity of water contained in the sand varied, dependent upon weather conditions and also upon the length of time which had elapsed from the time of dredging the sand to the time of deposit in the bins. The void tests of the sand therefore would vary considerably. To determine accurately the voids in the sand, a series of laboratory tests were conducted from time to time, and the voids in the Buck Island sand were found to run very uniformly at 34 per cent. The sand therefore was assumed by the inspector to have 34 per cent voids, thus eliminating the necessity of making a void test on the sand. This percentage was verified from time to time by laboratory tests, and no variation was found in the voids greater than 2 per cent, and in most cases this variation was much less.

The amount of water used in the concrete was under the direction of the inspector stationed at the point of placing the concrete, within the limits of the specifications. The inspector at the point of placing instructed the inspector at the mixer regarding the consistency of the mixture, and the inspector at the mixer regulated the amount of water to provide the consistency desired. The amount of water was measured for each batch, and this amount was varied to produce the proper consistency. In the hoppers also, the inspector at the mixer varied the amount of sand in accordance with the void tests on the gravel, and also allowed for the bulk of the sand when it was moist. At all times during the mixing of the concrete in the mixer, the inspector was able to look into the drum by means of a flood light, enabling him to observe the consistency of the mixture. This method of control of the mixture places the full responsibility for obtaining concrete that would meet the requirement of the specifications on the inspector at the point of placing. This method also permits those higher in authority from time to time to observe the quality of the concrete, particularly as regards the consistency, by noting

the depth to which the men track when working the concrete. A certain amount of shoveling of the concrete was necessary at the point of deposit, but for the most part the concrete was leveled off by a crew of men tramping and walking in the concrete.

In placing concrete in the larger areas, the 4-yd. bucket was used quite extensively. The 2-yd. bucket was used when depositing concrete near the forms or in other restricted areas. It has been found on this work that the 2-yd. bucket is the most efficient. The 4-yd. bucket presents a more difficult task in leveling off concrete and in thoroughly mixing it with the concrete already in place. The larger bucket, when depositing, leaves a

FIG. 2.—CREW SPREADING CONCRETE SHOWING DEPTH TO WHICH MEN TRACK WHEN WORKING THE CONCRETE.

cone of concrete which is too high, and one is never certain that the bottom central portion of this cone is thoroughly mixed except by leveling off this cone of concrete with shovels which takes considerable time.

The distance of transportation of concrete by railroad from the mixing plant to the point of placing was in some cases at least a half-mile, and some difficulties were encountered in the early part of the work because of the vibration in transit, causing the gravel to sink to the bottom of the bucket and the sand and cement to come to the top. This difficulty was overcome by adding 25 lb. of hydrated lime per cubic yard of concrete.

Mortar Tests.—In order that complete strength information might be had, tests were made of the concrete after it had been deposited. These

tests were accomplished as follows: At least once a week, samples of deposited concrete were made into briquettes to be tested for tensile strength, and also into 6 x 12-in. cylinders to be tested for compressive strength. Paper cartons were used for molding the cylinders. A sufficient number of briquettes and cylinders were made on each of these occasions to permit carrying these tests on over a period of five years. The briquettes were tested in the Hugh L. Cooper & Co. laboratory on the work, and the cylinders were sent to the Bureau of Standards for test. Table III shows a portion of the individual tests. Table IV shows the average of concrete

TABLE III.—SUCCESSIVE TESTS MADE SHOWING THE COMPRESSIVE STRENGTH OF WILSON DAM CONCRETE AND THE CORRESPONDING 28-DAY TENSILE STRENGTH OF THE MORTAR BRIQUETTES MADE FROM THE SAME CONCRETE BATCH.

(Six bags of cement were used per cu. yd. of concrete in the tests below.)

Date of Test	Compressive Strength (lb. per sq. in.)				Tensile Strength of Mortar Briquettes (lb. per sq. in.) 28 days
	28 days	3 mo.	6 mo.	1 year	
9-17-23	3,058	3,328	3,629	3,343	515
10- 5-23	2,089	2,723	2,389	2,265	411
10-13-23	1,906	2,542	2,419	2,199	435
10-18-23	1,518	2,094	1,963	2,014	350
10-26-23	2,954	3,724	3,235	3,463	459
10-29-23	2,561	3,059	3,115	2,909	500
11- 6-23	2,402	2,532	2,700	2,733	471
12-20-23	2,423	2,963	2,904	2,516	461
1-17-24	1,703	2,496	2,235	2,515	501
1-30-24	2,082	2,948	3,326	3,453	408
3- 8-24	2,855	3,178	3,857	4,043	431
3-14-24	2,595	3,320	3,323	3,606	469
3-22-24	2,103	2,683	3,012	3,027	345
3-31-24	1,706	2,356	2,787	3,468	402
4- 9-24	3,224	4,082	4,049	4,863	610
4-23-24	2,749	3,401	3,809	4,061	470
5- 2-24	3,006	3,801	4,275	5,176	543
5- 9-24	3,094	3,886	4,221	4,770	559
5-13-24	3,164	4,014	5,027	4,614	523
6-10-24	3,783	4,461	5,395	4,200	544

samples taken over a period from March 1923 to December 1924. The summary of Table IV shows for the 5-bag mix a gradual increase in the compressive strength from 1,613 lb. per sq. in. at 14 days to 2,645 lb. at six months, with a slight decrease to 2,631 lb. at one year. For the 6-bag mix over this same period, compression tests show a gradual increase from 2,137 lb. per sq. in. in 14 days to 3,351 lb. per sq. in at the end of one year.

In regard to the tensile briquettes for this same mortar, the 5-bag mixture shows an increase from 303 lb. per sq. in. at 7 days to 479 lb. at three months, with a slight decrease to 472 lb. at six months and another decrease to 454 lb. at one year. For the 6-bag mix, the tensile briquettes

show an increase from 384 lb. at 7 days to 533 lb. at three months, with a slight decrease to 504 lb. at one year.

The proportions of our standard mixed concrete average as follows: For the 5-bag mix, 1 part of cement to 2½ parts sand to 5 parts gravel; for the 6-bag mix, 1 part of cement to 1¾ parts sand to 4¼ parts gravel.

To show the effect of excess water in concrete a number of tests were made using our standard mix as a basis. Table V shows test No. 1 as our standard mix. Test No. 2 was the driest mix that could be used, and tests 3 and 12, inc., were made adding 5 per cent more water in each case, beginning with test 3, so that test 12 had the wettest mixture which had the consistency of a liquid. You will note that Table V shows that the compressive strength decreased with the addition of water. Our standard mix produced 3,559 lb. per sq. in. compressive strength at the end of one

TABLE IV.—RECORD OF TESTS OF CONCRETE PLACED FROM MARCH 1, 1923, TO OCT. 9, 1924.

(25 lb. of hydrated lime was used per cu. yd. of concrete.)

Amount of Cement Added per cu. yd. Concrete	No. of Concrete Batches Sampled	No. of Cylinders Tested for Each Average	Compressive Strength in lb. per sq. in. Cylinders broken by Bureau of Standards, Washington, D. C. — Age of Cylinders					No of Briquettes Tested for Each Average	Tensile Strength of Mortar Briquettes in lb. per sq. in. Briquettes made in the manner described in the test — Age of Cylinders					
			14 da.	28 da.	3 mo.	6 mo.	1 yr.		7 da.	14 da.	28 da.	3 mo.	6 mo.	1 yr.
5 bags	37	111	1,613	2,007	2,523	2,645	2,631	185	303	398	452	479	472	454
6 bags	44	132	2,137	2,560	3,154	3,343	3,351	220	394	439	505	533	512	504

year, and the wetter mixes in general showed gradual decreasing strength as the water ratio was increased, until test 12 gave a compressive strength of only 1,521 lb. The type of concrete used at Wilson Dam prohibited the placing of concrete by means of spouting, in fact it was necessary, in accordance with Colonel Cooper's specifications, to use the bucket method of placing of concrete, and hand shoveling when direct dumping from buckets was impossible.

Cleaning Old Concrete Surfaces.—The cleaning of each concrete surface before depositing the next layer is very important, and this was accomplished by removing the laitance and other undesirable material to a depth of from ½ to 2 in. It was found advantageous to remove a considerable portion of this laitance when the concrete was about twelve hours old by brushing the surface with push brooms. This removed the soft greasy laitance without disturbing any of the gravel, but was only effective when the concrete was green enough to be affected by the broom

and still not suffer deformation from men walking on it. After the concrete had become set sufficiently so that picks could be used the surface was gone over and any laitance not removed by the brooms was picked off. The method of testing the old concrete surface to determine if all the laitance had been removed was found to be best accomplished by moistening the fingers and then rubbing on the surface of the old concrete. If the laitance had been removed, the surface thus rubbed would present a gritty, rough touch to the fingers, but if any laitance was still present, a greasy or slippery feeling could be detected.

Very little trouble was encountered on this work because of frozen concrete, as the weather conditions in Alabama during the greater portion of the year are suitable for concrete work, it being only at occasional

TABLE V.—STRENGTH OF CONCRETE WITH DIFFERENT QUANTITIES OF WATER.

(Experimental batches—hand mixed; mix 6: 9½: 25.)
(No hydrated lime was used in this series of experiments.)

Test No.	Percentage of Water Based on Weight of Dry Ingredients	Water Ratio. Volume of Water to Volume of Cement	Total Amount of Water in gal. per cu. yd. of Concrete	Compressive Strength (lb. per sq. in.)			
				28 days	3 mo.	6 mo.	1 year
2	8.56	0.899	40.80	2,640	3,229	3,359	3,458
1	8.95	0.955	42.85	2,165	2,986*	3,070	3,559
3	9.22	0.976	43.95	2,285	2,650*	2,954	3,481
4	9.46	1.01	45.35	1,906	2,680	2,672	3,218
5	9.88	1.05	47.13	2,128	2,501	3,013	3,236
6	10.30	1.096	49.3	1,886	2,366	2,705	3,006
7	10.75	1.142	51.42	1,931	2,392	2,733	2,864
8	11.20	1.194	53.56	1,875	2,418	2,679	2,684
9	11.64	1.241	55.70	1,629	1,967	2,403	2,430
10	12.09	1.292	57.85	1,307	1,653	1,729	2,115
11	12.38	1.317	59.20	1,316	1,546	1,594	1,784
12	12.85	1.38	61.45	1,048	1,280	1,497	1,521

* Four breaks were averaged as one specimen was faulty. All other results in strength are the average of five 6 x 12-in. cylinders.

NOTE.—The cement used in the tests passed the specifications of the American Society for Testing Materials in all respects. The 7-day tensile strength of its 1:3 cement sand briquettes was 266 lb. per sq. in., the 28-day test 384 lb. per sq. in.

intervals during the winter that the thermometer goes below 32 deg. F. On such occasions freshly-placed concrete was protected over-night and thereafter for a period of 24 to 48 hours by covering with canvas and using salamanders or steam lines. In any case where the surface of old concrete or rock surface on which concrete was to be deposited showed indications of being very cold or had ice on it which would prevent a good bond of the concrete, this surface was cleaned and warmed by a jet of live steam.

Bonding Concrete Courses.—Before depositing concrete on any new surface the older surface was grouted with a thin layer of neat cement grout thoroughly brushed into the old surface by wire brooms. The surface of concrete being deposited was kept as nearly level as possible and the concrete was spread so as to keep the surface fresh at all times.

Expansion Joints.—In order to obstruct the flow of water through any of the vertical expansion joints or through any horizontal construction joints, key boxes, 10 in. deep by 20 in. wide, were placed vertically in the expansion joints and horizontally in the surface of each completed area of concrete.

Permeability.—In order that information might be obtained concerning the permeability of the concrete, flat cylindrical specimens 14 in. in diameter and 4¼ in. high were made from the concrete actually placed. These were cured in moist air for a period of from 30 to 60 days and then tested by applying a 100-lb. hydrostatic pressure on the 6-in. diameter interior surface area for a period of 40 hours. Table VI shows the average of several permeability specimens. It will be noted from this table that increasing the amount of cement made the concrete more dense and permitted less water to penetrate the concrete.

TABLE VI.—PERMEABILITY OF WILSON DAM CONCRETE.

(Tested at 101 lb. per sq. in.)

Amount of Cement per cu. yd. of Concrete	No. of Batches of Concrete Sampled	No. of Specimens Tested	Age of Specimens when Tested	Average Results in Permeability in cubic centimeters				Permeability in gal. per sq. ft. for 40 hours
				10 hr.	20 hr.	30 hr.	40 hr.	
5 bags*	5	5	1 to 4 mo.	1,000	1,482	1,822	2,079	2.81
6 bags*	5	5	1 to 4 mo.	693	989	1,230	1,414	1.91
6½ bags†	5	5	1 to 4 mo.	540	847	1,088	1,277	1.72

* Twenty-five lb. of hydrated lime admixture was used for each cubic yard of concrete.
† No hydrated lime was used.

Temperature Conditions in Concrete.—In the main structure of the dam, vertical expansion joints were carried from the rock foundation up to the top of the structure, so that at no point would the distance between expansion joints be greater than 46 ft., and in many cases the distance was less than this.

As the expansion and contraction of the concrete is affected by the temperature conditions existing in the mass, it was decided to investigate the temperature conditions by means of thermometers placed in the mass concrete. Electrical resistance thermometers were embedded in the concrete of one section of the dam, to determine the maximum temperature reached by concrete in the process of curing, and also to determine the length of time required for concrete to lose the effect of the heat caused by chemical action of the cement. In addition to this, information will be obtained showing the effect of seasonal changes in temperatures on the expansion and contraction of the concrete. The record being obtained from these temperature tests is not yet complete and therefore only general deductions can be made at the present time.

This equipment consisted of:

One resistance thermometer indicator; 2 dial switches of 24 points each; and 49 resistance thermometers with lead-covered leads.

The cost was approximately $1500.

The degree of sensitivity permits the temperature to be read to ¼ deg. F.

The resistance bulbs were placed in two vertical planes, one on the center line of the block, and the other 6 ft. from the expansion joint. Twenty-four thermometers were placed in each of these sections.

In all cases the thermometers were placed before the concreting was started in the lift in which they would be embedded. A hole was drilled

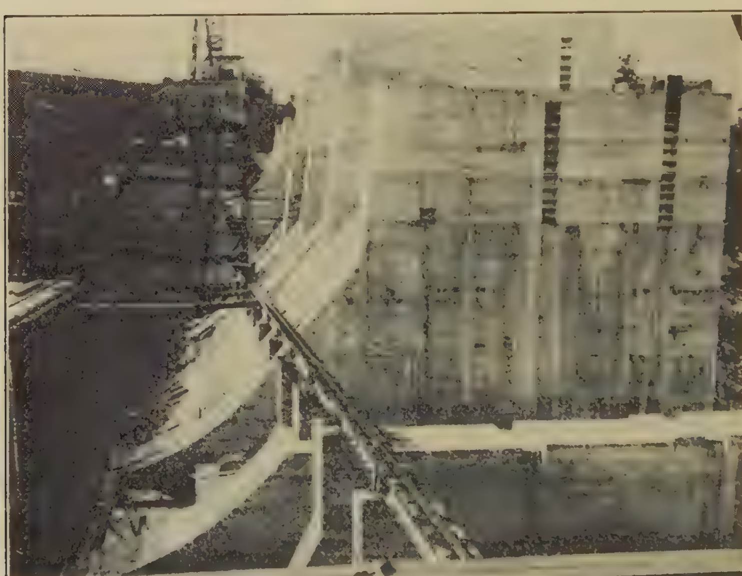


FIG. 3.—TYPICAL ARRANGEMENT OF KEY BOXES IN PIERS OF SPILLWAY SECTION.

into the concrete of the previous lift and a rod grouted into this hole. The thermometer was shoved through a 5-ft. length of ¾-in. steel conduit, so that the thermometer bulb projected about 4 in. out of the lower end of the conduit. The excess lead-covered cable was coiled at the upper end of the conduit. The 5-ft. length of conduit was then wired to the vertical rod, so that the thermometer bulb was at its proper elevation.

After the concrete had been placed and set, the conduit projecting above the concrete was then bent over and connected to the group of conduits leading vertically from the inspection tunnel in the body of the dam, where the switchboard and indicator were installed. This method

proved very satisfactory, as all 48 thermometers were installed without injuring in any way the thermometers or lead-covered leads.

Thermometers No. 7 and No. 31 were placed in recesses on the upstream face of the dam, to record the temperature of the water at different depths.

The scope of this report does not permit of detailed description of the action of each thermometer, but a study of the plates will give a considerable amount of information. Some of the general points of interest are:

Thermometers placed in the rock foundation 8 ft. below the concrete showed a gradual rise in temperature from about 67 deg. to 81 deg. F. during a period of about five months. Thermometers located 1 ft. from the surface showed a rise from 50 deg. to 83 deg. F. in five days, and then a fall to 43 deg. in 30 days, after which the temperature increased and decreased with the ambient temperature.

Thermometers located 2 ft. from the surface showed a rise of temperature from 64 to 82 deg. in 24 hours, and then a fall to 73 deg. in three days, at which time a new layer of concrete was placed and the temperature rose again to 86½ deg. in five days and then gradually fell till the temperature reached 56½ deg. at the end of two months.

Thermometers located 9 ft. from the surface, showed a rise in temperature from 63 deg. to 79 deg. in two days and then a fall to 70½ deg. in three days, when a new layer of concrete was placed, excluding the effect of the ambient temperature. The temperature then rose to 100 deg. in eleven days, after which it gradually decreased, reaching 76 deg. at the end of three months.

When a layer of concrete was placed, the thermometers were located 1 ft. below the surface, consequently the ambient temperature affected the thermometer until the next layer of concrete was placed, after which the thermometer recorded the temperature developed in the mass where it was located.

Thermometers located 20 ft. from the surface showed a rise of temperature from 64 deg. to 84 deg. in 24 hours, and then a fall to 76 deg. in three days, when a new layer of concrete was placed. The temperature then rose gradually to 101 deg. in 11 days, after which it fell to 78 deg. in five months.

Thermometers located 26 ft. from the surface showed a rise from 47 deg. to 113 deg. in 24 days, and then a fall to 81 deg. in seven months.

Information obtained from these temperature records to date shows that in large mass concrete the temperature near the surface increases lightly for a short period of time, approximately four or five days, and then falls slowly for as much as 30 days, after which the rise and fall of the temperature is directly affected by the ambient temperature prevailing. At greater depths in the concrete mass the temperature rises much higher, to as much as 113 deg. F., over a period of approximately three or four weeks, and then gradually decreases for a period of as much as eight or nine months.

It is interesting to note that this concrete was placed in the winter season when the ambient temperature would tend to decrease the maximum reached by the mass, and yet this maximum was 113 deg. Also, it is to be noted that concrete near an expansion joint, even though in contact with adjoining mass concrete, does not increase to the same temperature reached by the center of the mass, although this difference is apparently not over three or four degrees. The information shown by this record is that the central portion of any mass concrete is sometimes in a state of expansion while the external surfaces are in a state of contraction, and explains the cracks appearing in the surface of concrete which do not penetrate to the interior of the mass. The record of this temperature experiment is not yet complete, and therefore all the conclusions cannot be drawn at the present time, but it is evident from the information so far obtained that the interior of a mass of concrete 25 ft. or more from the surface is affected by the general seasonal change of ambient temperature, but is not affected by the daily changes. Also, the interior of the mass may be undergoing a change caused by a seasonal change of ambient temperature which occurred several months before.

Conclusions.—The testing of materials entering into the concrete is of great importance, and the method whereby all cement was tested twice has proven of great value.

The best information regarding the concrete being placed in the structure is obtained from the actual mortar tests from samples of the concrete after it has been placed.

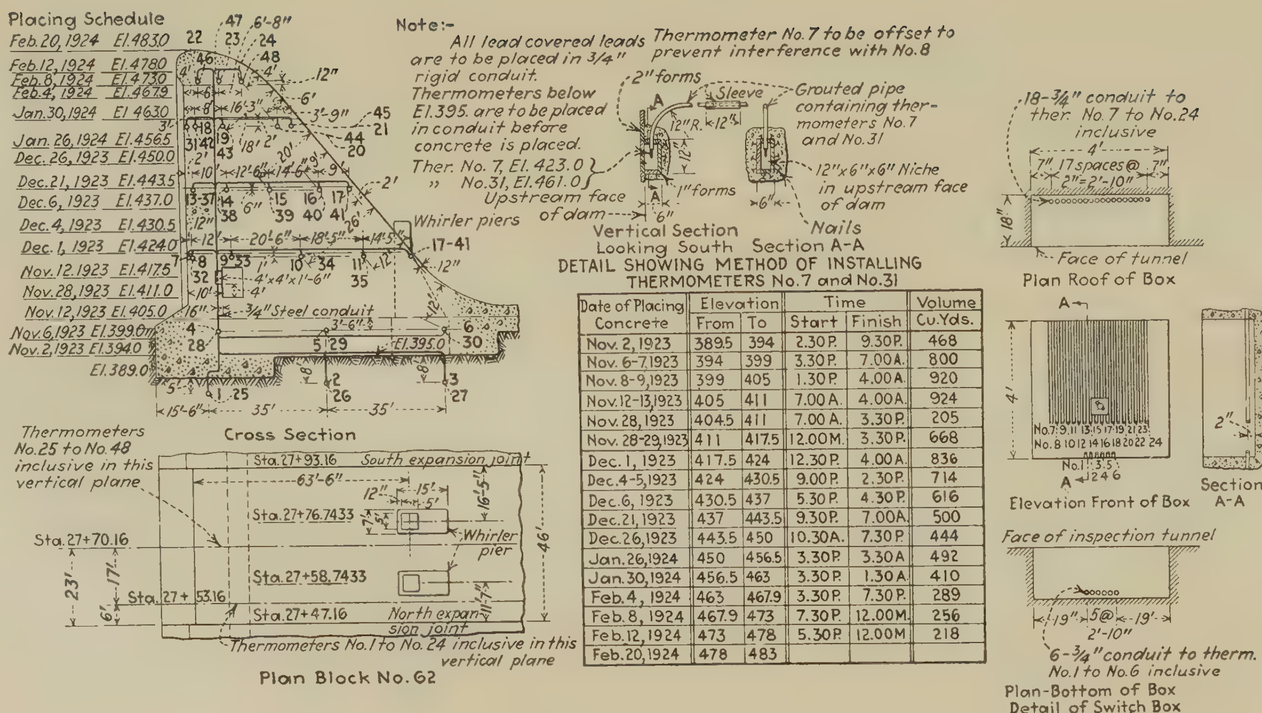
The method of controlling the concrete whereby the inspector at the point of placing has full charge and full responsibility for the quality of concrete so placed, controlling it so that the men shall not track deeper than 10 in. nor less than 2 in., is a very efficient and successful method.

Because of the high continuing temperatures in concrete of considerable volume, if cracks in permanent work are to be avoided, deposits of concrete should be limited to 4 ft. for any one deposit. Such deposits should be allowed to set and cool at least five days before succeeding courses are placed.

The greatest horizontal dimension for any given deposit should be 25 ft.

All deposits should be provided with horizontal and vertical key boxes wherein the area of the keyways is at least 5 per cent of the area receiving concrete.

Colonel Cooper believes we will obtain the desired quality in our concrete structures in this country only when we limit the amount of water used to the minimum amount which will permit satisfactory placing and working of the concrete in the forms.



Date of Placing Concrete	Elevation		Time		Volume
	From	To	Start	Finish	Cu. Yds.
Nov. 2, 1923	389.5	394	2.30 P.	9.30 P.	468
Nov. 6-7, 1923	394	399	3.30 P.	7.00 A.	800
Nov. 8-9, 1923	399	405	1.30 P.	4.00 A.	920
Nov. 12-13, 1923	405	411	7.00 A.	4.00 A.	924
Nov. 28, 1923	404.5	411	7.00 A.	3.30 P.	205
Nov. 28-29, 1923	411	417.5	12.00 M.	3.30 P.	668
Dec. 1, 1923	417.5	424	12.30 P.	4.00 A.	836
Dec. 4-5, 1923	424	430.5	9.00 P.	2.30 P.	714
Dec. 6, 1923	430.5	437	5.30 P.	4.30 P.	616
Dec. 21, 1923	437	443.5	9.30 P.	7.00 A.	500
Dec. 26, 1923	443.5	450	10.30 A.	7.30 P.	444
Jan. 26, 1924	450	456.5	3.30 P.	3.30 A.	492
Jan. 30, 1924	456.5	463	3.30 P.	1.30 A.	410
Feb. 4, 1924	463	467.9	3.30 P.	7.30 P.	289
Feb. 8, 1924	467.9	473	7.30 P.	12.00 M.	256
Feb. 12, 1924	473	478	5.30 P.	12.00 M.	218
Feb. 20, 1924	478	483			

FIG. 4.—LOCATION OF THERMOMETERS.

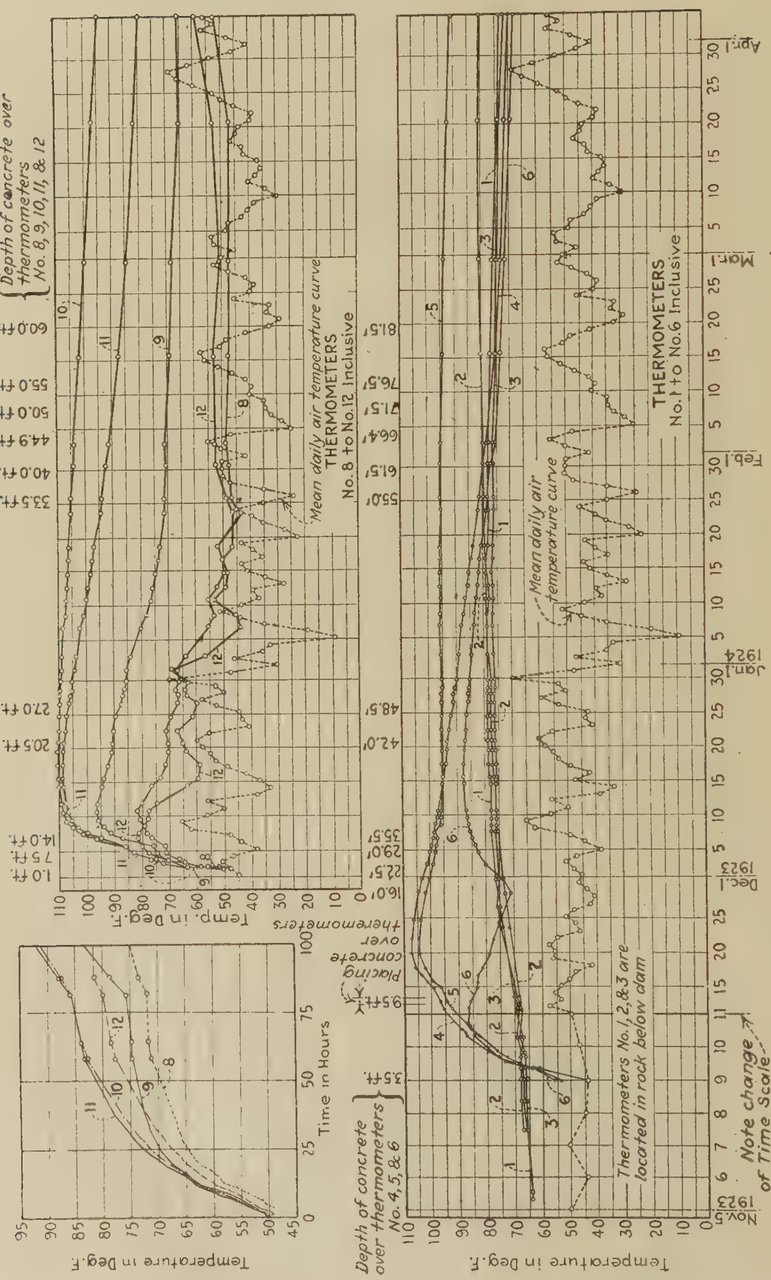


FIG. 5.—TEMPERATURE CHANGES IN MASS CONCRETE.

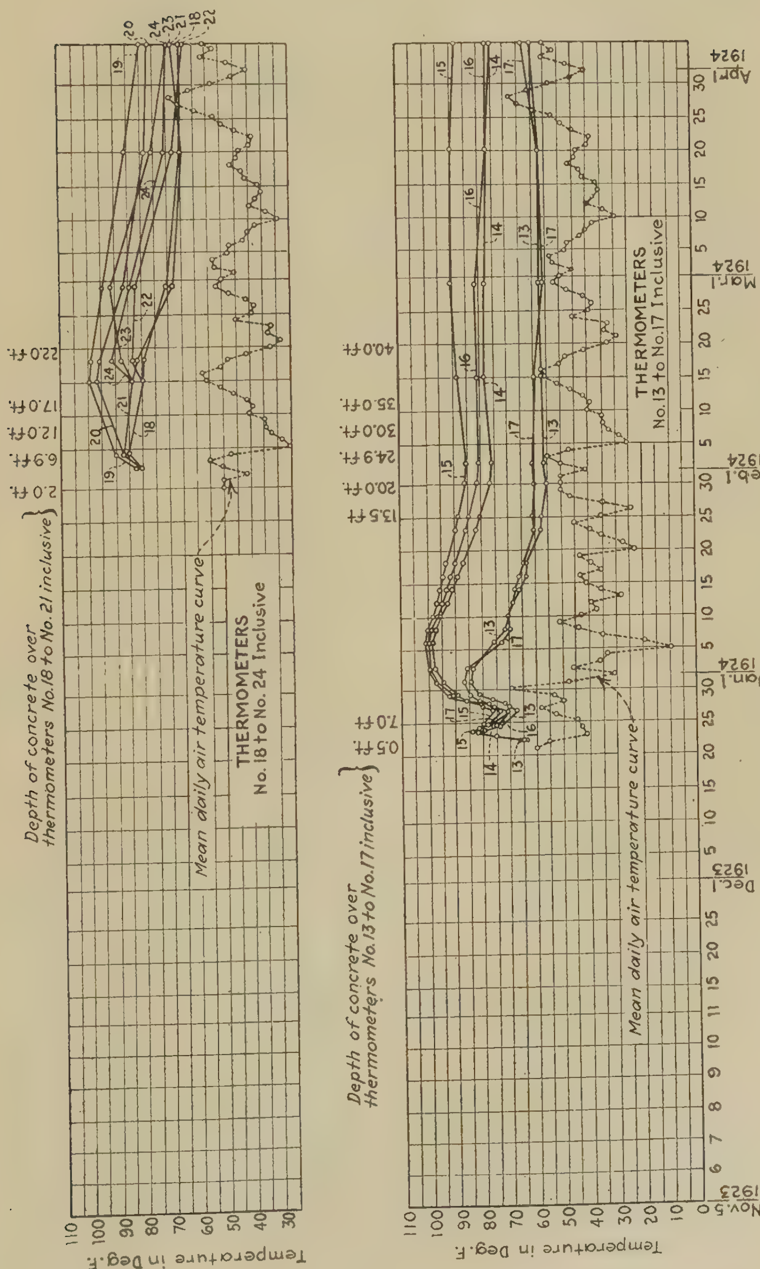


FIG. 6.—TEMPERATURE CHANGES IN MASS CONCRETE.

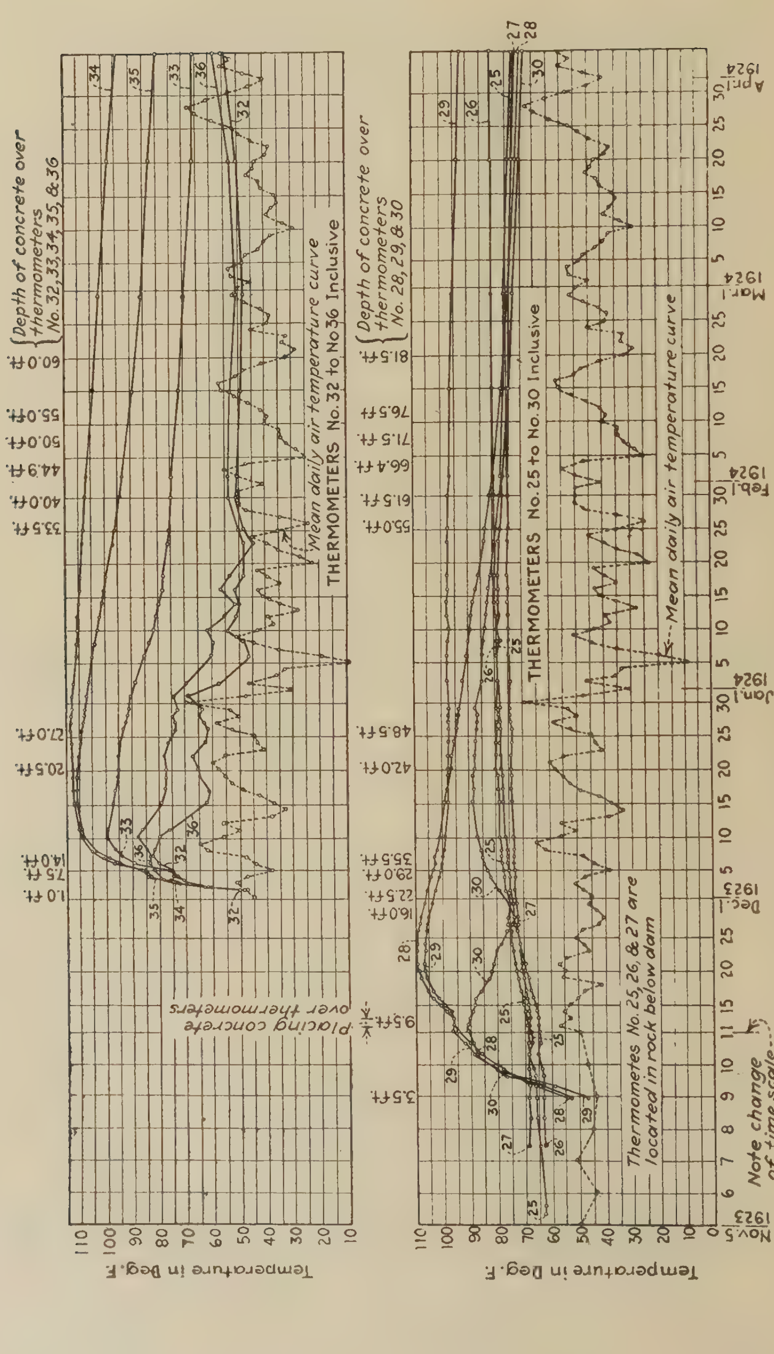


FIG. 7.—TEMPERATURE CHANGES IN MASS CONCRETE.

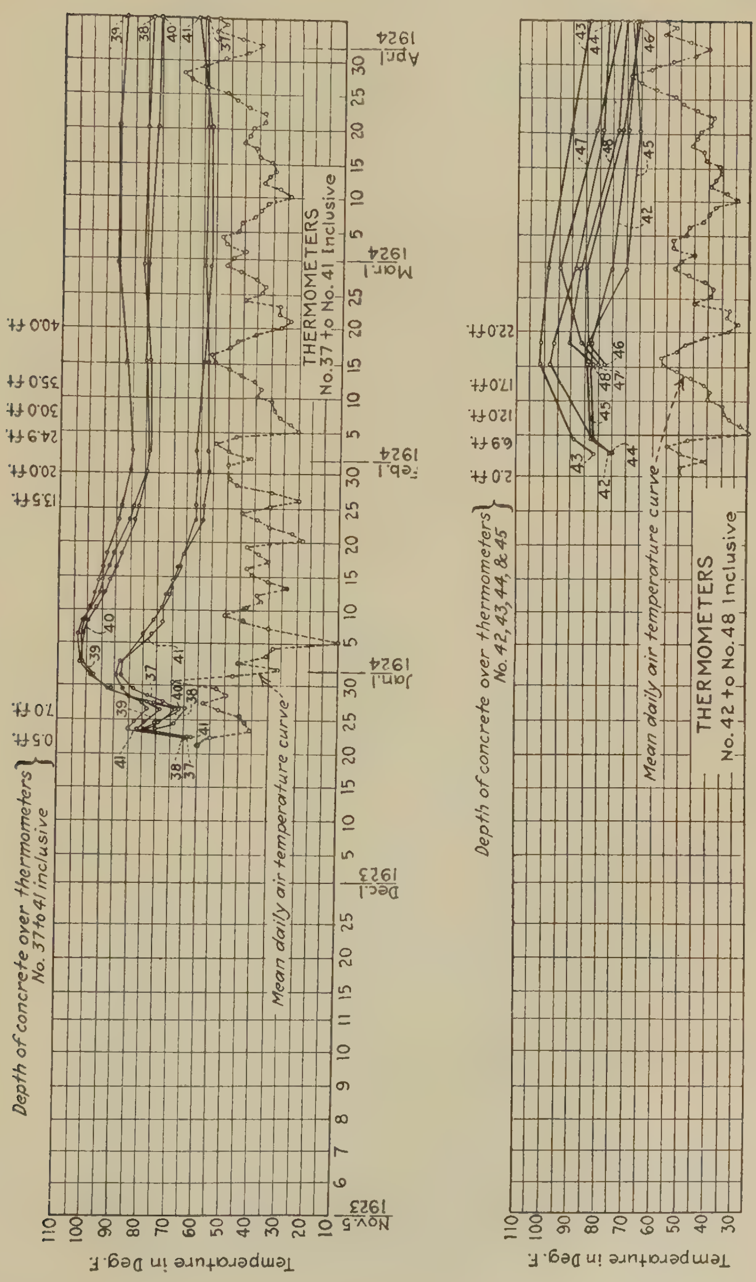


FIG. 8.—TEMPERATURE CHANGES IN MASS CONCRETE.

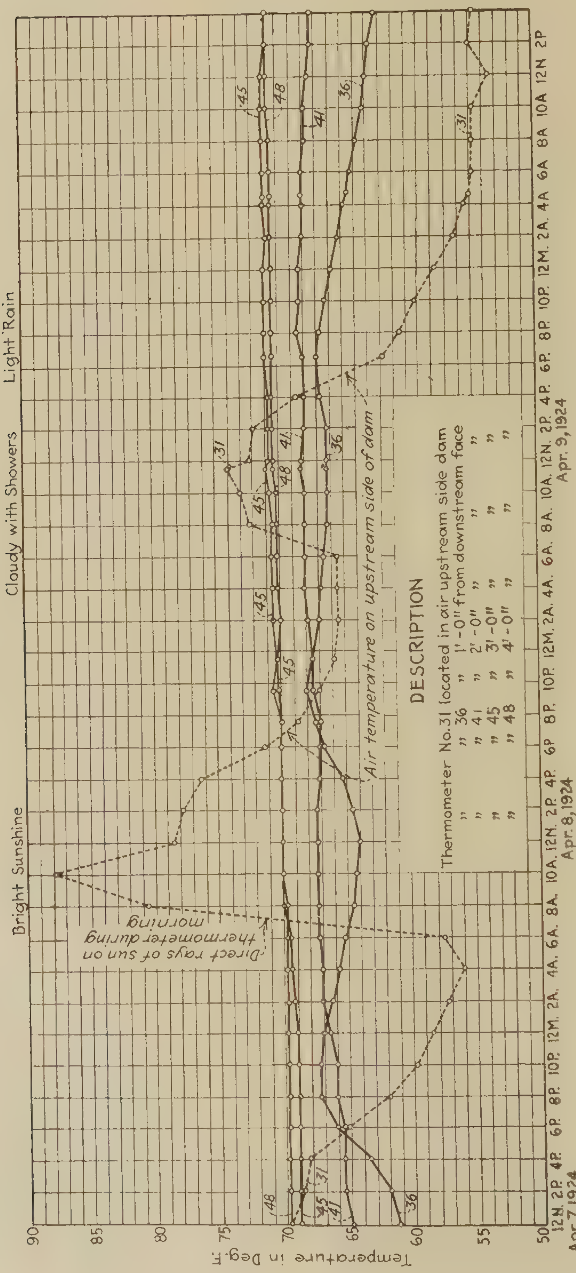


FIG. 9.—TEMPERATURE CHANGES IN MASS CONCRETE, DAILY.

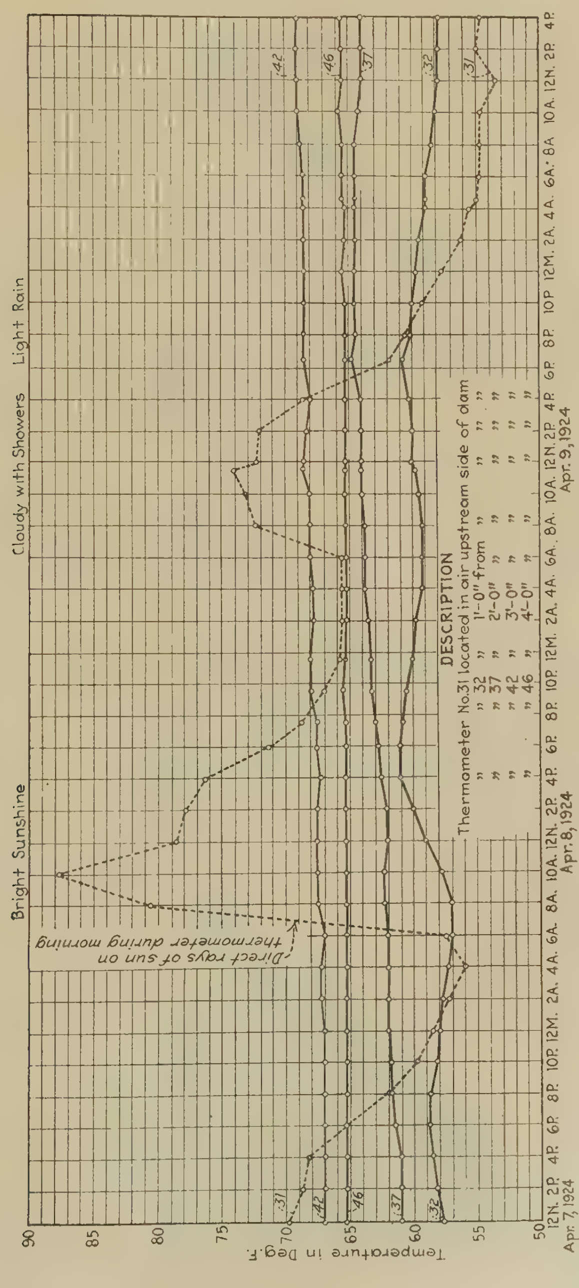


FIG. 10.—TEMPERATURE CHANGES IN MASS CONCRETE.

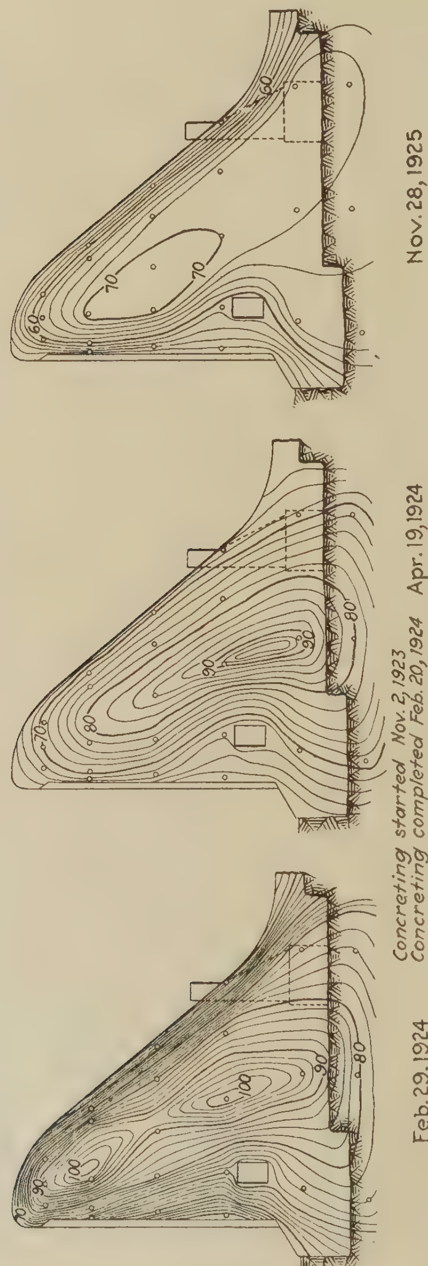


FIG. 11.—ISOTHERMAL CHART.

DISCUSSION.

Herbert J. Gilkey.—These are very valuable temperature records showing that interior concrete was not only subjected to a high temperature during the setting process, but probably to very high shrinkage stress as well. I wonder if any steps were taken to find out how this concrete will differ in quality from that in other parts of the structure. Also, I would like to know how those mortar strengths compared with the strength of the concrete as placed. **Mr. Gilkey.**

Mr. Hall.—I will answer first the question about the mortar test. Those tests of the concrete after it had been placed gave the average of a number of different brands of cement. Our tests prior to the placing of concrete were the testing of individual brands of cement only. The test afterwards was the test of the structure as it stood; consequently there is no direct comparison between the cement before and the concrete afterwards, except in a general way. The general result is that our mortar did test higher than the standard 1: 3. Our concrete is a bit richer; our mortar ratio is, in the case of the rich mix, 1 part cement and 2 1/5 parts sand, so of course we got a higher result in our mortar tests. **Mr. Hall.**

Mr. Gilkey.—I thought that the mortar was screened out from the concrete and that compressive tests were also made on the screened-out mortar specimens. Therefore I thought that we might have data giving a direct comparison between the compressive strength of a concrete and that of its mortar. **Mr. Gilkey.**

Mr. Hall.—The mortar briquets were made as follows: A sample was taken from the concrete in a bucket. Stones ¼-in. or above were taken out and the mortar thus left was made into briquets. Another bucket was taken from that concrete and cylinders were made for compressive strength, leaving all the gravel in. That mortar that you referred to represents our mortar tests of the job; that was our essential test showing the strength of the concrete. It shows the result of that testing where the tensile strength was 400 lb. in 28 days. **Mr. Hall.**

Mr. Gilkey.—No compressive test of the mortar? **Mr. Gilkey.**

Mr. Hall.—Of the concrete, but not the mortar. **Mr. Hall.**

(In answer to the question as to whether any instrument was placed in the interior of the concrete to measure the amount of expansion or contraction, Mr. Hall replied that this was not done. Numerous measurements were made at expansion joints on the surface but no interior measurements.)

Mr. Chapman. MR. CHAPMAN.—How were the samples of the gravel or coarse aggregate taken to determine the variation in voids? Attention has been called to several cases of the difficulty of taking a sample of coarse aggregate that will be truly representative of the bulk of the shipment, and I would like to get the information on the method for insuring the sample being truly representative. I would be interested to know whether the difference was noted in the strength result, either the cylinders made from concrete in place or the penetration of the workers was 2 or 10 in. Of course we cannot tell very definitely what the water-cement ratio is which the 2-in. or the 10-in. penetration represents. Was there any effort to record the penetration of the tube at the time of taking samples or to find the difference between the 2-in. and 10-in. penetration?

Mr. Hall. MR. HALL.—In regard to the question of the men walking in the concrete, the leeway of having them track to the depth of 2 to 10 in. was permissible for the following reason. In summer weather it is very hot and the concrete dries more rapidly; in winter weather when the weather is cooler and the concrete takes longer to set up, it is more advantageous to use the drier mix at all times. We made the effort to use the driest mix possible which would produce good results. There was no effort made to compare the 2-in. trackings with the 10-in. In other words, the 10 in. was the limit to which the inspector could go. He was given that much leeway to produce good results. Probably our average concrete ran 4 to 5 in. in tracking. You noted that the men were standing about ankle deep and that would be 4 or 5-in. tracking and represented a fair sample of the tracking method. Ten inches was not permitted except under extreme circumstances and on a hot day or when, for some other reason, it was difficult to keep the concrete fresh.

About a measurement of the voids: In the sand, the measurement of voids naturally would be very difficult by the man at the mixer, because of the amount of moisture contained in the sand under different conditions. If the train had just come from the dock where the sand was very moist and the inspector measured that, he would get an untrue result, and the loading of the sand into the bin was made sometimes simultaneously from the car and from the stockpile. The sand in the stockpile would be drier than that coming directly from the car. The large amount of concrete used in any run of 24 hours meant that the bins were frequently completely empty during the day, and sometimes several times, so that the aggregate would vary. Therefore, we made what I refer to as the laboratory test on the sand; we took numerous samples of sand to the laboratory, thoroughly dried them out, getting rid of all the moisture and then found out the amount of that type of sand to use, and those tests were made at intervals so that the inspector at the mixing plant was relieved of the necessity of making a void test on the sand.

In regard to the gravel, the same difficulty is not encountered as the gravel will contain a certain amount of moisture and does not vary as

much as the sand under moisture conditions. The gravel void measurements were made at least once an hour by the inspector at the mixer, and were checked by a representative of the testing laboratory, who kept that result.

However, there is one feature in regard to this method of control to which I wish to call your attention; the inspector at all times was able to look into the mixer on the front side by means of flood lights. These inspectors became so proficient that when they saw the gravel come down into the hopper, they could tell whether the gravel was changing and if so vary the amount of the sand and could make a void test to verify the result and invariably they found they were in the right direction. The general result was that the void tests in the gravel were made at the mixing plant but the void tests in the sand were made by laboratory measurements to prevent errors creeping in through measuring sand which might be damp or might be dry.

FRANK H. JACKSON.—What was the maximum variation in the voids of the coarse aggregate? **Mr. Jackson.**

MR. HALL.—That is a question I would have to answer somewhat from memory, the voids in the coarse gravel were about 47 per cent, in the sand the voids were about 34 per cent. **Mr. Hall.**

MR. JACKSON.—You do not recall the variation in the coarse aggregate? **Mr. Jackson.**

MR. HALL.—The voids were 44 per cent to 50 per cent. **Mr. Hall.**

ARTHUR A. LEVISON.—What was the range in the moisture content of the fine aggregate as it was batched at the mixer on the job, approximately? **Mr. Levison.**

MR. HALL.—Sometimes it went as high as 20 per cent moisture, by weight. It varied all the way from probably 7 per cent to about 20 per cent moisture. **Mr. Hall.**

STEPHEN STEPANIAN.—Was the gravel you received partly crushed or round, and was there a variation between the time you received the crushed gravel and the round gravel? There are certain banks in which one finds fine gravel and at other times it is crushed. If the gravel were crushed there would be a variation of voidage, I imagine, approximately between 50 per cent and 41 per cent from angular to round. **Mr. Stepanian.**

MR. HALL.—The gravel ran very uniform, due to the fact that it was dredged from the bed of the river. The dredge was moved up and downstream and that gravel was found to run very uniform. None of the gravel was crushed. Any boulders, which sometimes were obtained and which might run as large as one's head or one's fist and would not go through the screen, were simply wasted. At times crushed limestone rock was used. There was no effort made to run either one thing or the other. In the early part of the job we had crushed rock, sometimes we had gravel and sometimes we had a mixture of the two. We simply took our void tests of either kind or of the mixture. The bulk of the job, though, 95 per cent of all the concrete, had a gravel content. **Mr. Hall.**

Mr. Colburn. D. S. COLBURN.—Did you find that the tensile strength of the mortar would correspond with the compression strength on the concrete? Was there a relationship there which could be traced?

Mr. Hall. MR. HALL.—In general there is a relationship, although we did find in some cases more retrogression in the briquettes than the cylinder. I consider the compressive strength by the cylinder method a more reliable test of the concrete than the tensile briquettes.

Architectural Concrete.

By John J. Earley.*

A discussion of architectural concrete naturally touches on its nature, its artistic value, its economic value, the proper manner of using it and the best means of making it. Such a subject cannot be treated in a paper of reasonable length, therefore; I have selected just two thoughts for your consideration.

I have been guided in my selection by the suggestions of others. I received the suggestions from the tendency of every one, in accordance with his training and his interests, to accent some phase of the subject. Architects seem to be particularly interested to know the proper manner of using architectural concrete, the preparations to be made for it, the restrictions imposed by it on other materials. They want to know how to fit architectural concrete to a building. Makers of concrete products want to make concrete to measure up to architectural requirements. They have heard that the control of water in concrete helps to do this, and they want to know what water does in concrete and the means of controlling it.

It is common knowledge that, all other things being constant, the control of concrete is exercised through the control of the water in it. This has frequently been discussed before the American Concrete Institute, but the discussion has been restricted to the relation of water to the strength of concrete. The relation of water to the other properties of concrete, the properties which distinguish one concrete from another and befit each for its particular use, has not been discussed. Furthermore the function of time in the control of concrete has not to my knowledge been considered by the Institute. I think that this should be done because the use of the function of time in the placing of concrete and the hydration of portland cement requires the exercise of human judgment which is craftsmanship.

I hope that the control of concrete, or craftsmanship, or technique or whatever best expresses the art of making concrete will be constantly kept before the Institute.

Permit me to illustrate the manner of using architectural concrete by telling how the architects for the Louisiana State University and for the Nashville Parthenon solved the problem.

There is in Louisiana an old tax—the Severance Tax. It accrues to the State whenever its natural resources are exported. It never attracted much attention until with the discovery of oil unexpected wealth

*Architectural Sculptor, Washington, D. C.

flowed into the treasury of the state and it became the duty of the governor to invest the wealth for the people. This was a difficult and serious problem for the governor, who being a man of vision decided to invest in the young men and women of the state and to build for them a university. The governor hopes that Louisiana will gain from this investment a profit beyond measure, the development of the natural resources of the state by its own sons and daughters.

The old agricultural and mechanical college was used as a nucleus around which to build the other departments of a modern university. A new site was selected and new buildings and equipment were planned. An architect was chosen to design and construct a group of buildings to adequately house the university and to express the dignity of the ideals and the hopes with which the work had been undertaken.

It is not a derogatory criticism but merely a statement of fact to say that the state of Louisiana does not afford the character of craftsmanship necessary to the satisfactory execution of an architectural design in keeping with the dignity of this project. But the state of Louisiana does afford the labor and material suited to sound construction.

Here, then, is the problem which confronted the architect—should he build with the labor and materials at hand a group of buildings which from a purely utilitarian view point would be structurally sound and useful but which in their appearance would fall far short of the architectural possibilities? On the other hand, should he for the sake of appearance import into the state craftsmen and materials suitable for the work? It seemed to be a pity to lose so fine an architectural opportunity but at the same time it seemed unreasonable to reject or slight the labor and material afforded by the state. If labor and material from home and abroad could be used independently and at the same time, a solution might be found which would certainly be expedient.

This problem reacted upon the designing of the building and complicated it to a considerable degree. The importance of the undertaking indicated buildings of the first class but in the circumstances buildings of that class were hard to get. Then, too, this university is in a warm climate and certain types of buildings suitable to colder climates would not be suitable to it.

Complicated problems often have simple solutions. It was true in this case. It became immediately apparent in the search for a suitable style of architecture that the South could lay claim to a style of its own. In the South that style of building, which is generally spoken of as a "mission" building, is accepted and used for first-class structures. A mission building was originally the expression of someone's memory of buildings of the domestic type in Spain or Italy. Mission buildings have been well suited to the use and climate of the South, therefore, the architects decided that the buildings of the Louisiana State University should be a domestic type of the style of building developed in Italy during the Renaissance. It was desired that these buildings should be an intuitive

course in architecture for the students, and that they should take home improved standards in architecture to be reflected in the architecture of the state when these young people in turn began to build. That this hope might be well founded, simplicity, permitting intimacy, encouraging emulation and making the buildings a part of the life of the people, was preserved.

A complete separation of the finish from the structure was the solution of the problem. It permitted the architect to construct with the labor and material afforded by the state and to finish with a better class

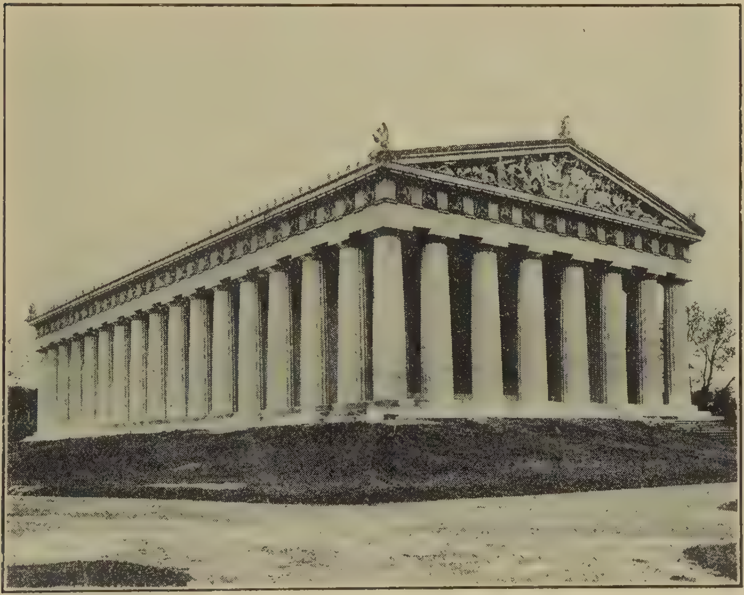


FIG. 1.—THE NASHVILLE PARTHENON.

This is a replica of the Athenian Parthenon. It is both architecturally and archeologically an educational building of the first order.

of labor and materials imported for that purpose. In this also a similarity exists between these buildings of today in Louisiana and those of the Renaissance in Italy. For, the latter also were built with the labor and materials at hand. The people took stones from the fields and built them into walls, rough, irregular, unsightly and none too weatherproof. They refined these walls and made them symmetrical, beautiful and weatherproof with a separate finish, a stucco.

This idea of a separate finish is again exemplified by the restoration of the Nashville Parthenon.

One of the chief attractions of the Tennessee Centennial Exposition

held in Nashville in 1897 was the Arts Building. It was in size and drawing a pleasing representation of the Athenian Parthenon, which is regarded as the culmination of the architectural skill of the Greeks. The body of the Nashville building was of brick, the columns, entablature and tympani being of staff on wood lath. After lasting twenty years the gypsum plaster naturally fell into ruin. But the beauty of the building had endeared it to the people of Nashville and the Board of Park Commissioners decided not only to reconstruct it in permanent material but

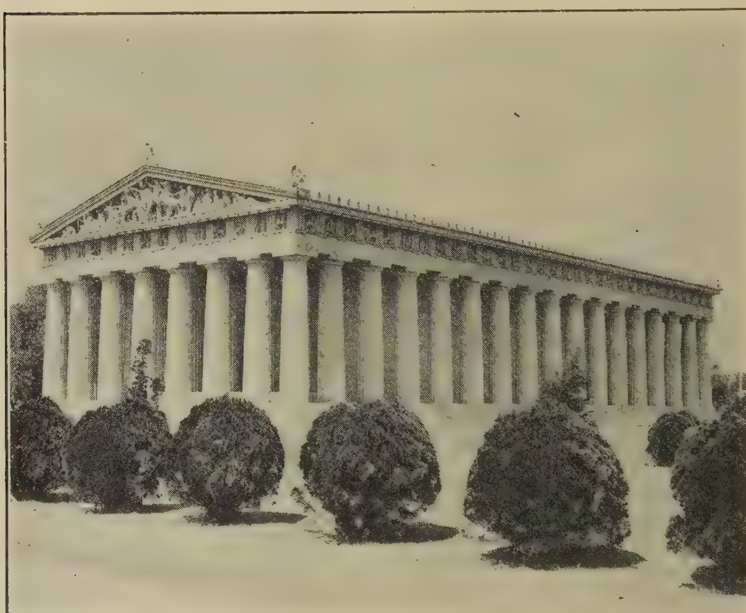


FIG. 2.—THE NASHVILLE PARTHENON.

Though not posed as was the original but its beauty is sufficient to overcome the disadvantage of a low setting in an open park. The building has a soft reddish yellow color not unlike the present color of the ruins. The polychrome of the entablature follows the Greek conventions and is probably the same as it was on the original.

to make it a reproduction of architectural and archeological accuracy. They instructed their architect to include the best known of those remarkable ingenuities which Greek architects employed to correct certain optical effects. Among these for example are entasis or swell which corrects the concave appearance of a straight column (the swell of the Parthenon columns is about ¾ in.), a sense of stability received from the inclination of columns (the inclination of the columns of the Parthenon is about 3 in. in 34 ft.), entasis which corrects the apparent sag in a long hori-

zontal line, which is aggravated when a number of vertical lines rest upon it. (The curvature upward of the through lines of the Parthenon is about 2.6 in. on the short sides and about 4.4 in. on the long sides.)

The Parthenon at Athens was of Pentelic marble throughout but the cost of such material in the reproduction was prohibitive. The means by which the original had been built were no longer economic and other more economic means had to be devised. The architect faced the problem of accurately reproducing certain optical sensations by new means, of reconstructing the Parthenon with modern material.

FIG. 3.—THE SCULPTURES HAVE BEEN RESTORED FROM THE ELGIN MARBLES AND FROM THE BEST AUTHORITIES.

The Eastern pediment represented the birth of Pallas Athena who sprang forth full armed when Vulcan cleft the head of Zeus.

A study of ancient Greek architecture seems to warrant the conclusion that the Greeks used marble as a building material, because it was the most generally satisfactory material within reach, and because in the hands of fine craftsmen it could be made to take the form and to receive the finish which to the Greeks were standards of beauty. The belief seems to be justified that marble was not considered to be a complete medium, because they applied a separate finish to it. They painted it. The Greeks did not like the glare of marble but they did like colored surfaces and detail. Even sculpture developed in the latter stages of Greek art was painted.

FIG. 4.—VIEW OF THE WESTERN PEDIMENT.

It is equally well established that the Western pediment tells the story of the rivalry between Pallas Athena and her uncle, Poseidon, for precedence in the land of Attica. Here are pediments 90 ft. long and 14 ft. to the apex filled with heroic figures and cast in concrete.

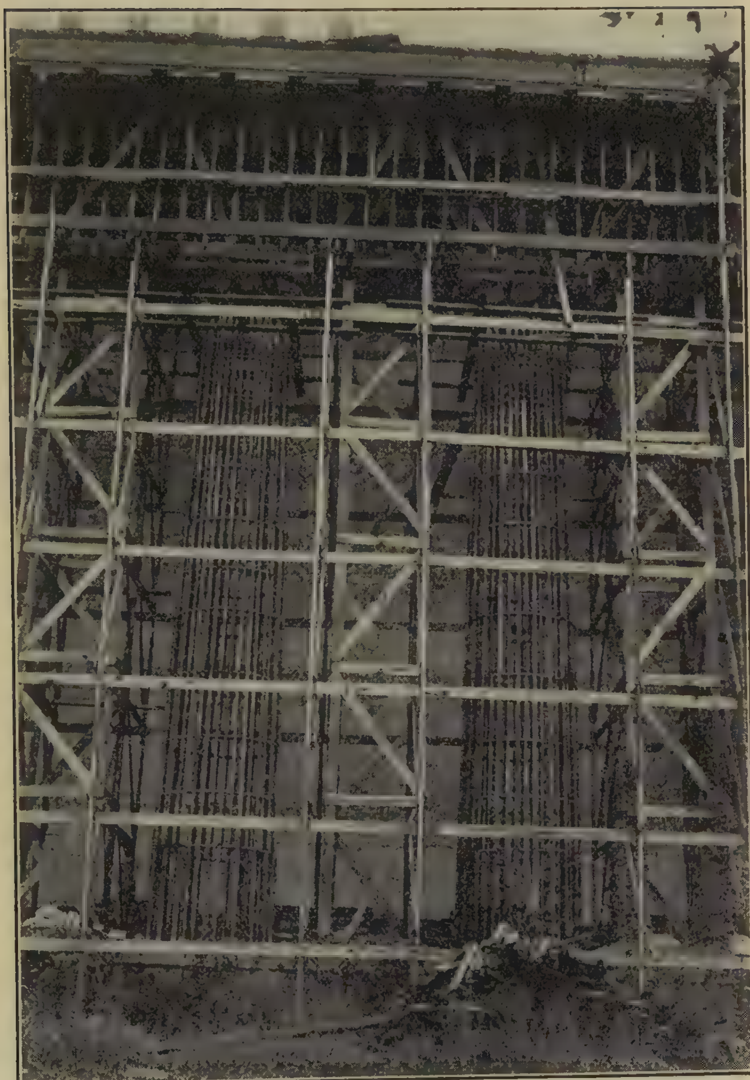


FIG. 5.—COLUMN RECONSTRUCTION.

The old Fine Arts Building of the Tennessee Centennial fell into decay, and we were commissioned to restore it in a permanent material. The construction of its mighty columns was the real adventure of the job. It was decided that structural cores should first be built and that a finish should be applied to them.

FIG. 6.—CLOSEUP OF COLUMN RECONSTRUCTION.

The outside wall of the mold was constructed with vertical staves and spacing blocks held together by encircling rods. Metal lath was attached to the inside edges of the staves.

FIG. 7.—COLUMN RECONSTRUCTION OF NASHVILLE PARTHENON.

The inside wall was segmental in form. It could be taken apart and removed through the top of the column. Thus a porous mold was constructed which separated the excess water from the concrete at the proper time. The time required for the cement to hydrate permitted a different consistency to be used in the two phases of the concrete. It was mixed with enough water to facilitate the placing of it. Then the free water was withdrawn leaving the concrete in an almost ideal state to develop greatest density and strength.

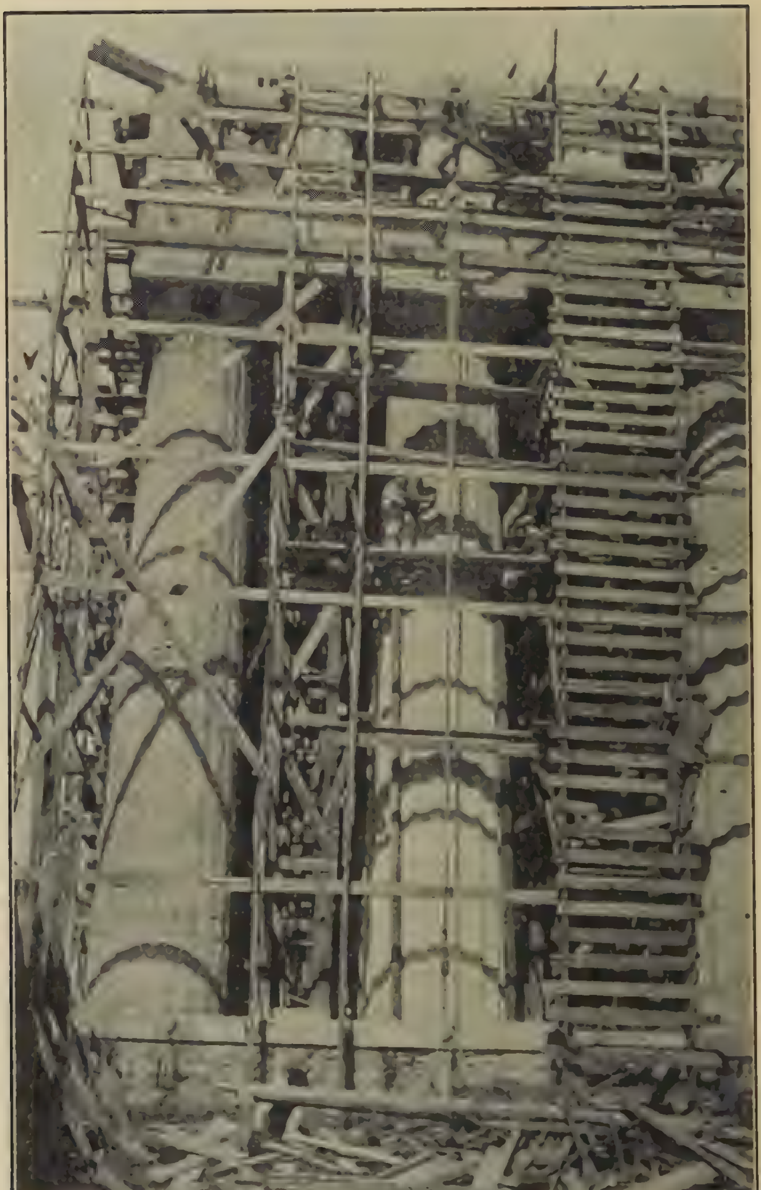


FIG. 8.—COLUMN MOLD REMOVED.

When the mold was removed the metal lath remained on the concrete, later it also was stripped off. With it came the skin of fine cement usually found on concrete. This left the structural cores both porous and rough, an ideal state for the attachment of the finish.

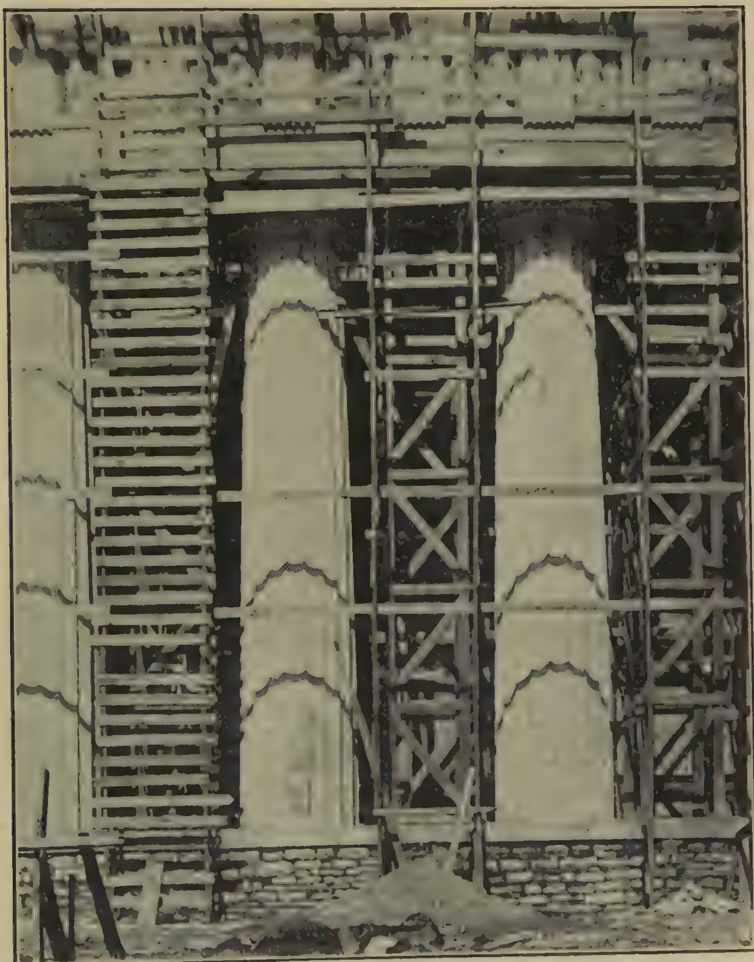


FIG. 9.—REBUILDING THE PARTHENON ENTABLATURE (NASHVILLE).

The entablature was constructed on the same principles. It was a U-shaped beam over the columns and was cast in porous molds made with metal lath. Around all these structural elements molds of the finished form were assembled and into them was poured the finishing concrete. Precast details of exquisite workmanship were added.

FIG. 10.—AN INDICATION OF COLUMN SIZES.

It is difficult to judge the size of objects in a picture. So, Professor Abrams offers to show you just how large these great columns really are.

The practice of applying the finish separately is well illustrated by the Baths of Caracalla (Marcus Aurelius Antonius Bassianus) in Rome. To me this example is excellent. Together with the Parthenon it is sufficiently important to be included in the text of architecture. It was an engineering problem of the first magnitude built as such with suitable labor and materials. It was an architectural project of equal importance

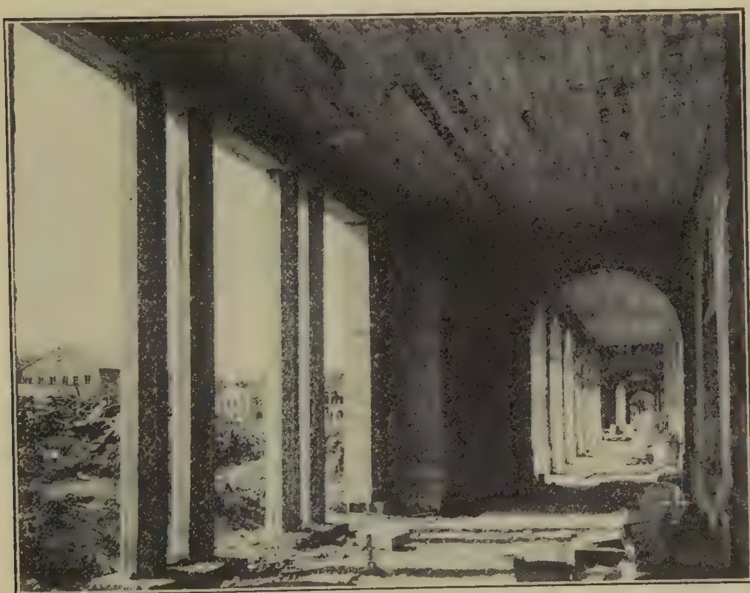


FIG. 11.—DETAIL OF A LOUISIANA STATE UNIVERSITY STRUCTURE SHOWING THE CONCRETE STRUCTURE ON WHICH THE FINISH WAS APPLIED.

Rome surely must be the haunt of many great spirits; architects, engineers and craftsmen. They have left the record of a manner of building which completely separated the finish from the structure. Old walls made with brick and concrete still show the holes of bronze dowels which once held their marble finish to them, and gaunt ribs once held the lineaments of beauty until they were picked to build medieval Rome. The methods with which these ancient glories were built were applied to the building of the Louisiana State University. Its buildings were made with brick and concrete and the finish was completely separated from the structure.

handled as such with fitting craftsmanship and media. The two undertakings were not confounded.

In those days in Rome laborers, craftsmen, artists and even architects were mostly slaves taken in war. So, when such a work as this was undertaken it was organized for the best use of available resources. There were unskilled hands a plenty and into them was put a material suited to their use, the long rough Roman brick. With this they built, bearing the burden of the work, while skilled hands, of which there were relatively

few, decorated floors, walls and vaults with form and color now treasured among the achievements of our race.

I recommend to architects a complete separation of finish from structure. I am convinced that this is the best manner of using architectural concrete. Perhaps it is the best manner of using all decorative materials. It permits all classes of labor and material to be used to advantage. It separates skilled from unskilled labor and fine from coarse material. There are in the building industry many splendid organizations fully equipped to construct but not to decorate, who are unwilling to undertake a project in which the architectural appearance is the major consideration. It is an advantage to employ such specialists and it is little trouble to separate their work from the decorations. I have known the measure of this advantage, the saving a specialist in construction will make from the cost and time of another contractor, to almost equal the cost of a well-designed finish in architectural concrete.

This completes the first thought of the paper. It stresses the complete separation for economic reasons of the finish from the structure during the operation of building. The second thought of the paper on the control of water in architectural concrete products follows.

I am convinced that "mixing water" means more to a craftsman than to an engineer. I have received the impression that engineers regard water as an important element in concrete to be measured and mixed with as much care as the cement and aggregate and to be kept in a somewhat constant relation to them in volume, particularly to the cement. They seem to think that there are four principal things to be considered, namely; the measuring and mixing of ingredients, the placing of the concrete in forms, the testing of the hardened concrete and the reproportioning of the ingredients to correct all defects. If this be generally true, and I think it is, it would seem to indicate a belief that whatever may be wrong with concrete is the result of something wrong with the proportioning of the water, cement and aggregate. The expression "too much mixing water" has become trite as an explanation of defects in concrete. I cannot subscribe to this belief, because I know, though very imperfectly, some of the things which happen in concrete while being mixed, while being handled, while hardening and finally after hardening. A recognition of what occurs in concrete in each successive phase and of the time when the concrete passes out of one phase into another is necessary to the proper control of water at the proper time. Thus, the control of water at the right time, is the major principle in the technique for making architectural concrete.

Craftsmen in other arts recognize the function of time in the control of other media and the makers of concrete and concrete products should do so. Do not misunderstand me: I am not saying that the control of water in time is not practiced in making concrete, for, such a neglect would prevent the making of concrete. But I say that time should be recognized as a factor and used with better understanding.

FIG. 12.—LOUISIANA STATE UNIVERSITY BUILDINGS.

Plain walls of homely materials speak loudly of economy. They were built with the labor and materials at hand but without embellishment.

FIG. 13.—LOUISIANA STATE UNIVERSITY BUILDINGS.

Simple in design and simple in construction they were built for the needs and to the liking of a simple people, but they possess with all a dignity worthy of their purpose.

FIGS. 14 AND 15.—LOUISIANA STATE UNIVERSITY BUILDINGS.

When finished they assumed a refinement of form and color which is very pleasing. They became a course in architecture to the students. From them can be separated a motif for any ordinary building, a house, or a business block. The refined simplicity of the domestic style of the Italian Renaissance has been well expressed.

There is much information on this subject in the plastic arts, in the records of the scientific societies and the research laboratories. For example—Technologic Paper of the Bureau of Standards No. 43, The Hydration of Portland Cement, records a study of the effect of various amounts of

FIG. 16.—LOUISIANA STATE UNIVERSITY BUILDINGS.

Concrete has made this possible in a country where the like could not have been done by any other means.

water and steam on the individual components of portland cement and on portland cement itself.

In the summary of that paper two out of five pages are devoted to the time in which the various ingredients of portland cement become hydrated

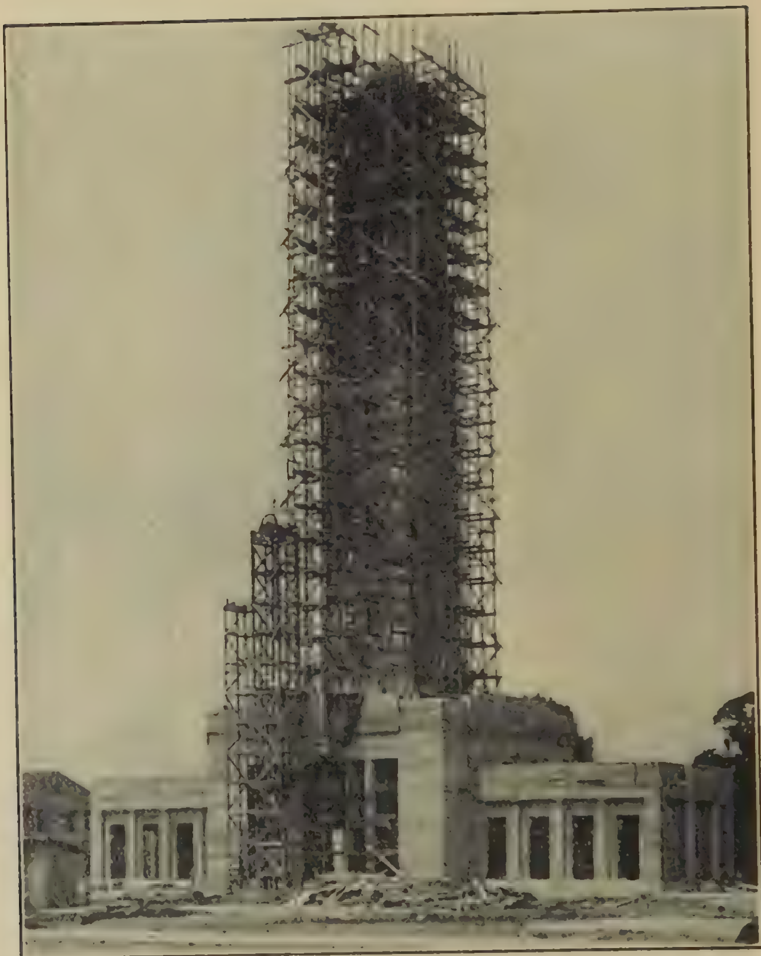


FIG. 17.—CAMPANILE UNDER CONSTRUCTION.

The Memorial tower with its clock and chimes and with its crowning lantern was built with reinforced concrete, steel and tile, wherever each served best.

in water. I can frankly say that when I took up the study of portland cement as a binder in plastic media after years of study and experience with other binders such as gypsum, lime and clay, this paper encouraged me to an heretical disregard of the dogma of initial set, and to a belief in

FIG. 18.—THE COMPLETED CAMPANILE AT L. S. U.

Over all, architectural concrete spread a skin of beauty colored with soft yellow, red and white like the campanile of an old church.

my own freedom to do anything I might reasonably wish to do to portland cement in the first twelve hours after wetting it.

In the *Proceedings* of the American Concrete Institute, Vol. XVII, 1921, there is a paper on "Shrinkage of Portland Cement Mortar and Its

Importance in Stucco Construction." The studies reported were undertaken to determine the causes of crazing in stucco and because volume changes capable of causing crazing could occur in the first twenty-four

FIG. 19.—A BREATH OF THE RENAISSANCE.

Like a glimpse of old Italy to teach the boys and girls of Louisiana that familiar things may still be pleasing.

hours, measurements of them were made before and during the set of the mortars, as well as afterward. The paper clearly indicates successive phases in the making of mortars. These may broadly be divided into

two, namely, when the material is plastic and when it is solid. With equal clarity the paper indicates twelve hours as the approximate time of the first phase.

In the art of plastering the technique of white coating or finishing offers another example. As the finishing material is spread over the rough mortar or under coat, part of the mixing water slowly leaves it passing into the absorbtive base. The plasterer waits until just the right amount of water has passed before he trowels the material to a hard smooth finish. He uses time as a factor in the control of a plastic material.

In many arts the control of material through water has been studied and well conducted scientific investigations have enriched the arts with new and better knowledge. Some of this work has been published but much of it has not. What has been published is widely scattered through the literature of many industries, such as felt, paper, glue, ceramic, gypsum and lime industries. But I think for our present purpose it may be summarized as follows: Water exists in concrete in more than one state; in some we can work with it and in some we cannot. There is water needed to make crystals and jellies, this we may not touch because to do so would destroy the cement. There is water very tightly held in pores and on surfaces, with this we can do little because we lack strength to move it. There is water free in the mass, with which we may do whatever is needed, if we know how.

You will remember that years ago concrete was mixed with very little water and tamped into place with much labor. It was hand-made and was good concrete. Experience and research confirm the fact that concrete made with just enough water is the strongest concrete, and in other respects is the best. When the forms became more complex and labor more expensive this kind of concrete was not economic. It was decided then that we were not handling concrete in the proper way. The consistency was changed from plastic to fluid and with fluid concrete difficult forms containing complicated reinforcements were better filled and with less labor. Mechanical devices were used and through them, with water as a vehicle, concrete was placed more easily than by hand. It was known that added water would decrease the strength of concrete but it would still be strong enough. The new methods accented the facility of the material and the desirability of taking full advantage of that property.

I suggest that all our formulas for water are a laudable effort to keep the flow in concrete and to put back the strength which was impaired when the flow was first added. Furthermore, to complete the second thought of this paper I suggest another step in technique. The introduction of time. As we have about twelve hours in which to work and as we can easily control the free water: let us deliberately add enough water to concrete to produce an optimum consistency for placing, and when the concrete is in the forms let us remove enough of the water to produce the best consistency for the strength and other good qualities of the concrete,

FIG. 20.—FACULTY HOUSES—OUTPOSTS OF THE IMPOSING WHOLE.

Around the University Group are scattered the faculty houses. They are fitting outposts to the principal group.

FIG. 21.—THE PRINCIPAL UNIVERSITY GROUP.

This consists of twenty-seven major buildings and occupies more than one-quarter of a square mile.

Soaps as Integral Waterproofings for Concrete.

By Alfred H. White* and John H. Bateman.**

The effect of soaps added in small amounts as integral waterproofing to cement mortar and concrete is to diminish the absorption of water by capillary action to a marked extent and to lessen the permeability to water under pressure. The compressive strength is not materially affected provided the waterproofed product is kept damp continuously until it gains the requisite strength and provided the soap is not one which causes foaming in the concrete mixer with entrainment of air in the finished product. If concrete containing soaps is allowed to become dry before it has attained the desired strength, it will always remain weak because, being waterproofed, water will not be able to gain free access to the interior of the mass and continue the hardening process.

Introduction.

In the manufacture of concrete, good cement and water, aggregates of proper quality and gradation of size, and proper mixing are fundamental. Other substances in rather wide variety have been proposed as additions to the concrete during the process of mixing. The term waterproof itself needs to be defined for our purpose, for very few substances ordinarily considered as waterproofings are completely effective. Even varnish applied in several coats does not prevent the absorption of moisture by wood as is shown by the way many doors swell and stick in the humid weather of summer. A membrane built up of several coats of felt cemented by proper bituminous material is almost completely waterproof and is one of the few methods of complete waterproofing applicable to concrete, for the membrane if properly made is elastic enough to bridge minor shrinkage cracks. This paper discusses a type of waterproofing which is less effective than the membrane method but which has real value under many conditions.

As has been shown by one of us in several papers (1, 2, 3, 4) cement after reacting with water becomes, in part at least, colloidal and retains for many years and perhaps indefinitely its property of absorbing water.

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** Professor of Highway Engineering, Louisiana State University.

This absorption of water causes expansion and the evaporation of the water in dry air causes contraction. These volume changes which seem due to a fundamental property of portland cement are the most potent causes of the destruction of concrete exposed to the weather, as has been explained in some detail in the references cited. If concrete is to be kept constant in volume its moisture content must be held relatively constant.

One of the frequent reasons for desiring to waterproof concrete is to ensure a dry basement. A concrete wall to be satisfactory for this purpose must not allow water or even dampness to penetrate it and must have satisfactory strength. There are two quite different factors which may cause moisture to penetrate a concrete wall: pressure and capillarity. If a capillary glass tube like that from a thermometer, which is clean and has both ends open, is dipped into water the water will rise in the tube because of capillarity. If the walls of the capillary are even slightly oily the water will not rise in the tube. But if pressure is applied, water will be forced through the tube in spite of the oily surface, which was effective in preventing the rise of water due to capillarity. All concrete when dry contains capillary openings due to shrinkage of the colloid of the cement. Therefore a concrete surface in contact with water or wet soil will absorb water through capillarity and the water will diffuse into the concrete through the same force. In rich and well made concrete the penetration may proceed only a few inches before the colloid near the wet surface becomes swollen to such an extent that the capillaries become closed and further access of water is stopped. If the concrete is lean or not well made, the capillaries of the concrete will not become sealed by swelling and the water will penetrate through the concrete. If the concrete has large voids or is cracked, water may flow through it in a stream as through any other channel. Rich cement mortar and concrete in thick masses if free from cracks should therefore be waterproof after their colloid is well developed without any addition of waterproofing materials.

This resistance to water even in rich concrete comes only after proper curing and until that stage is reached even rich mortars are permeable to water. Rich concretes and mortars unfortunately have greater volume changes and are therefore more liable to crack because of changing moisture in their environment. There is an evident field of usefulness for a waterproofing which will prevent the penetration of moisture by capillary action even if it does not wholly prevent the penetration of water under pressure.

Classification of Integral Waterproofings.

Integral waterproofings may be classified as follows:

Inert—such as finely ground sand;

Water Absorbent—such as clay and hydrated lime;

Water Repellant—such as hydrocarbons and insoluble soaps.

The inert materials listed above are merely finely ground aggregates and are effective in proportion to their ability to make a dense concrete with a minimum of voids, although substances like ground slag do react in part with the cement. Clay and hydrated lime are both materials which absorb water, swell when wet and shrink when dry and behave in general as typical colloids. One of us has elsewhere discussed[5] the value of these materials but it is sufficient here to state that they promote the absorption of water by capillarity, rather than prevent it. Hydrated lime has a specific value, evident only after some months and under special conditions, due to the formation of calcium carbonate. Hydrocarbons such as petroleum residues have a definite waterproofing value but are difficult to incorporate and tend to leach out from the concrete.

Soaps are compounds of organic fat acids and inorganic bases. The soaps of soda, potash and ammonia are soluble in water, and those of lime, alumina and most other bases are both insoluble and water repellant. Ordinary toilet soap is a sodium soap. If it is used in hard water, containing calcium salts in solution, the insoluble lime soap is precipitated. The materials supplied commercially for waterproofing concrete consist either of the soluble or insoluble soaps. The soluble soaps dissolve in the mixing water and react with the lime set free as the cement hydrates, to form the insoluble lime soap. The insoluble soaps, being water-repellant, must have some ingredient incorporated to make them miscible so that they will become uniformly dispersed in the mixing water and remain in a very fine state of dispersion until the concrete has set. Since they are solids and insoluble in water they remain as minute water-repellant particles throughout the concrete preventing the absorption of water due to capillarity. Their value is a function in part of the particular kind of soap, but in larger measure, of its degree of dispersion in the concrete. Soap which is in large flecks will have much less value than one which is finely divided and uniformly distributed.

Query has been raised by some writers as to the permanence of "organic" compounds. The insoluble soaps of the saturated fatty acids are extremely resistent to the ordinary influence of decay. When animal remains decompose in the ground the fat acids resist decay for many years and form the adipocere which is sometimes dug up after all of the other animal tissues have disappeared.

Review of Literature on Use of Soaps as Integral Waterproofings.

The literature on the effect of soaps as waterproofings is, as was to be expected, somewhat conflicting, especially with regard to the effect of waterproofings on the strength of concrete. Tests by W. K. Hatt[6] made in 1904 showed that the effect of alum and soap solution added to the concrete mixer did not greatly affect the strength but diminished the absorptive power about 50 per cent. No quantitative figures were given by him. Baker and Derick[7] reported in 1908 on the effect of alum and soap and the following quotations are taken from their report:

"The most interesting feature of these experiments is that the alum and soap compound equal to an average of 1.2 per cent of the cement in a mortar containing an average of 23.7 per cent voids, stopped 65 per cent of the percolation: or, in other words, adding water-repelling void-filling material equal to approximately 5 per cent of the voids reduced the percolation to one-third of that of untreated mortar.

"The mixing of alum and soap in the concrete reduces its strength somewhat; but there are many situations in which strength is unimportant, or at least is less important than water-tightness. The effect of the alum and soap upon the strength of the mortar varies a little with the method of storing the test samples. For example, the mean of six neat portland cement briquettes mixed with 21 per cent of water which contained ½ of 1 per cent of alum and 1½ per cent of new "Ivory" soap, when left in the molds 1 day and stored in a moist chamber for 6 days, had a strength of 87 per cent of that of similar briquettes made with water alone; and when left in the moulds 1 day and then stored 6 days in water had 84 per cent; and when twice as much alum and soap were used, the strength was 83 and 71 per cent respectively."

Burton[8] reporting on tests made at Iowa State College in 1908 stated that soap and alum solution gave satisfactory results in waterproofing concrete pipe under pressure of a head of water of 100 feet and that tensile tests indicated that the soap and alum did not affect the strength of concrete.

Schick[9] writing in 1911 of the waterproofing of the foundations for a large flour warehouse in Budapest, stated that 6.6-8.8 pounds of soap dissolved in the water used for 1 cubic yard of concrete gave entirely satisfactory results.

Wegmann[10] on the other hand reported that though various integral compounds showed good waterproofing qualities they all seemed to reduce the tensile strength.

Grittner[11] has stated that with a mix of 1: 1.5: 2.5 by weight using water containing 8 per cent by weight of ordinary soap, an entirely satisfactory waterproofing was obtained with a reduction in the crushing strength at 28 days from 83.2 Kg. per cm² for the concrete without soap to 75.0 Kg. for the concrete with soap.

Burchartz[12] showed that a 1: 2: 5 concrete could be made watertight if soft soap to the extent of 1 per cent of the weight of the dry materials were mixed with the mortar. Compressive tests made on 1: 3 mortar with the same amount of soap added showed that the strength of 7 and 28 days was only about 40 per cent of that which contained no soap. A later report by Burchartz issued from the State Testing Laboratory of Prussia[13] showed that smaller amounts of soap solution were effective. Using 2 lb. of ordinary soap to 100 lb. of water the following crushing strengths were obtained at 28 days. The figures are given as percentage strength relative to a normal mixture.

1 : 2 mortar	crushing strength	93 per cent
1 : 3 concrete	" "	117 " "
1 : 2½ : 4 concrete	" "	98 " "

Hoffman[14] describing tests made in Germany in 1912 states that concrete in the proportion of 1 : 2.5 : 5 could be made water-tight if soft soap was dissolved in the water. He used 1 lb. of soft soap to 100 lb. of concrete which is about equivalent to 7 lb. of water. This quantity of soap reduced the compressive strength to about 40 per cent of the normal at 7 and 28 days.

Wig and Bates[15] reported on the elaborate tests made at the Bureau of Standards in 1911. They very properly state in their summary, "The addition of so-called 'integral' waterproofing compounds will not compensate for lean mixtures, nor for poor materials, nor for poor workmanship in the fabrication of the concrete." They go on to state that "the inert integral compounds (acting simply as void-filling material) are added in such small quantities they have very little or no effect on the permeability of the concrete." Permeability was measured in their tests by the amount of water passing through a disc of mortar 2 in. thick under a head which was usually 20 lb. per square inch although in some of the later tests, higher pressures were also used. The specimens were stored in a damp room between the time of molding and the time of testing, and were sprinkled three times each 24 hours. The specimens from a 1 : 4 mix showed so little permeability that we do not reproduce any figures. From the test on the 1 : 6 and 1 : 8 mortars we have selected the permeability at the end of one hour's exposure to the hydrostatic head. The original tables give figures for other times but space prevents quoting them in full. The best measure of the small quantity of actual active waterproofing material used is gained from the percentage of fat acid present. We have calculated this from the analyses given by the Bureau of Standards and include the figures in the tables. It will be a matter of astonishment to note that these figures for actual fat acid incorporated in the mortars are, in every case but one, less than 0.1 per cent of the dry mix. And yet their effectiveness is evident, although not necessarily in proportion to the amount of fat acid present. As previously explained the degree of dispersion of the insoluble soaps is perhaps more important than their quantity.

Table I gives the actual figures for the permeability of the 1 : 6 mortar. All of the test pieces allowed a material amount of water to pass on the two weeks test, but all of those containing waterproofing soaps made a better showing than the unwaterproofed mortar. At the end of four weeks the permeability was lower in every case than at two weeks due to the progressive development of the colloid of the cement during the hardening process. All but two of the waterproofed specimens were superior to the straight unwaterproofed mortar. Tests after 13 weeks in the damp room showed improvement for all mixes, but the unwaterproofed specimens were again at the bottom of the list and the specimens containing four of the

nine waterproofing compounds were reported as merely moist, and not allowing any water to pass. The tests at 26 weeks do not give any figures except for the longer tests but again the plain mortar was clearly the most inferior. Two groups with waterproofings were entirely impermeable, the surface of each of their three test pieces remaining entirely dry during a test of 24 hours. Three of the other waterproofings attained a rating of "moist" after 24 hours while 4 allowed water to pass in smaller amounts

TABLE I.—EFFECT OF INTEGRAL WATERPROOFINGS CONTAINING INSOLUBLE SOAPS ON PERMEABILITY OF 1: 6 MORTAR; FROM TECHNOLOGIC PAPER 3, BUREAU OF STANDARDS (TABLE 13).

Figures show tenths of a cubic millimeter of water passed per minute per square centimeter of surface subjected to 20 lb. per sq. in. hydrostatic pressure. The figure reported is the average for three test pieces of one minute readings taken at the end of one hour.

The symbol "m" indicates that the lower surface of the test piece became moist but that no water passed through. The symbol "0" indicates that the surface of the test piece remained entirely dry.

Waterproofing Compound No.	% Fat Acid Contained in Mortar	Permeability when pieces tested successively at: 2 wks.	4 wks.	13 wks.	26 wks.*	52 wks.*
None	0.0	1263	250	29	13(7)	3(24)
32	0.11	1190	277	m	11(7)	1(24)
31	0.008	1130	263	5	m(24)	
33	0.018	619	237	m	8(12)	m(24)
30	0.008	448	92	8	m(24)	0(24)
29	0.021	737	137	8	0(24)	
37	0.013	355	58	5	0(24)	
34	0.05	456	155	m	5(7)	
35	0.0007	421	95	3	m(24)	
36	0.04	400	68	m	3(24)	

* The figures in parenthesis indicate the number of hours during which the pressure was applied.

than the unwaterproofed specimens. Four groups retested at 52 weeks showed progressive improvement and again the unwaterproofed made the poorest showing. Table II gives the results for the 1: 8 mortar reported in a manner similar to that of Table I. Everyone of the results with the waterproofed mortars was better than those with the plain mortar at 2 weeks and 4 weeks. At 13 weeks and again at 26 weeks only two out of the nine waterproofings showed more poorly than the plain mortar. At 52 weeks they were all nearly impermeable. The unwaterproofed samples and

five of the waterproofed were reported as moist and of the other two one was entirely dry and one permitted a small amount of water to pass.

Wig and Bates, in the same report, described tests on the damp-proofing qualities of these same compounds made on discs of the same size as the test pieces for permeability. These discs were stored in the damp room for 21 days and then in the air for 7 days when they were tested by placing them in relatively shallow water to see how far the water would rise by capillary action. The method of testing measured only what might be termed the break-down when water rose so high by capillarity that the surface became damp. The specimens were two inches thick. With the

TABLE II.—EFFECT OF INTEGRAL WATERPROOFINGS CONTAINING INSOLUBLE SOAPS ON PERMEABILITY OF 1: 8 MORTAR; FROM TECHNOLOGIC PAPER 3, BUREAU OF STANDARDS (TABLE 15).

Figures show tenths of a cubic millimeter of water passed per minute per square centimeter of surface subjected to 20 lb. per sq. in. hydrostatic pressure. The figure reported is the average for three test pieces of one minute reading taken at the end of one hour.

Waterproofing Compound No.	% Fat Acid Contained in Mortar	Permeability when pieces tested successively at ages shown : 2 wks.	4 wks.	13 wks.	26 wks.	52 wks.
None	0.0	3343	1000.	87.	55.	moist
32	0.08	2290	566.	50.	16.	0
31	0.006	2790	632.	68.	37.	moist
33	0.014	2270	671.	71.	11.	moist
30	0.006	1890	606.	95.	3.	moist
29	0.016	1680	500.	147.	3.	moist
37	0.01	1790	566.	48.	moist	5
34	0.04	2870	548.	113.	55.	
35	0.0005	2098	429.	55.	79.	
36	0.03	1851	198.	moist	0	

1: 4 mortars all specimens remained sound for many days when immersed in a half inch and also in an inch of water. When, however, they were immersed in 1½ in. of water which brought the level of the water to within a half inch of the top of the test piece, the discs without waterproofing failed in 58 days. All of the waterproofed discs were still sound at that time and all but one remained sound for the whole period of observation which in most cases was 186 days. The tests with the 1: 6 mortar did not make such a favorable showing, only 4 of the 9 waterproofings yielding specimens superior to the unwaterproofed specimens. With the 1: 8 mortars 6 of the 9 waterproofings gave better results than the non-waterproofed specimens.

A further set of tests in this same series by Wig and Bates was made by taking the cubes which had been stored in the damp room for 28 days, immersing them in water for 28 days and determining the increase in weight. They state, p. 55, "However, the gain in weight after this 28 days' immersion was found to be less than 1 per cent of the original weight when first taken from the damp room, showing that the test pieces had absorbed nearly the maximum amount of water in the damp room." This is as was to have been expected since the cubes had been stored where they could not lose any of their initial water.

The tests of Wig and Bates as summarized above seem to indicate that their soaps exerted a distinct influence in preventing penetration of water. It is therefore rather astonishing to read in the Summary of their report (p. 85):

> "None of the integral compounds tested materially reduced the absorption of the mortars before they were dried by heating at 212 deg. F. Thus they would have little or no practical value."

The results of the compressive tests of the blocks in this same series of tests have been analyzed by one of us in a previous paper[5]. However we have no reason to dissent from the conclusions expressed by Wig and Bates, p. 85:

> "The addition of any of the compounds tested to a mortar in the quantities used in these tests does not seriously affect the compressive or tensile strength."

Abrams[16] has stated in a brief note his conclusion that the compressive strength of concrete was invariably reduced by all of the waterproofing compounds tested by him when stored in a damp room until tested after 28 days. He says:

> "Without exception the compressive strength of the concrete was reduced by these compounds. A soap solution of ¼ lb. per gallon of mixing water gave a concrete strength of 57 per cent of normal; 5 per cent crude oil, both asphaltic and paraffine base) gave concrete strengths 73 per cent of normal.
>
> Four different proprietary waterproofing compounds, used in the percentages recommended by the manufacturers, gave concrete strengths 74 to 87 per cent of normal; average about 80 per cent. A so-called alkali-proofing compound (patented) gave concrete strength 16 per cent of normal."

The only compound containing soaps which can be identified in the above quotation is that where ¼ lb. soap per gallon of mixing water is used, and this is a high amount.

This review of the literature has shown substantial testimony in favor of the effectiveness of soaps as waterproofing compounds. The differences of opinion are mainly as to the effect on the strength. Hatt, Wegmann and Abrams state that the strength was weakened by the soaps but do not

state the amount of soap used. Burton found the strength was not weakened but did not give figures as to the amount of soap used. The others all gave data which permits an approximate calculation of the quantity of fat acid in the finished mortar or concrete and the results are summarized in Table III. Apparently the products were all kept damp until tested. The figures show rather unexpected agreement in indicating that if the amount of fat acid in the finished product was less than 0.2 per cent there was little diminution in the strength. Attention should, however, be called again to the fact that these products were kept continuously damp until tested.

TABLE III.—SUMMARIZED EFFECT OF SOAPS ON COMPRESSIVE STRENGTH OF MORTAR AND CONCRETE.

Authority.	Per cent of Fat Acid in Mortar.	Effect on strength expressed in percentage of strength of plain concrete.
Hoffman	0.6	40
Buchartz	0.6	40
Burchartz	0.6	40
Grittner	0.5	90
Baker and Derick	0.36	71
	0.18	84
Burchartz	0.12	93-117
Schick	0.1	Satisfactory
Wig and Bates	0.2 and less	No effect

The Effect of Conditions of Initial Storage upon Strength of Concrete with Soaps as Integral Waterproofings.

The investigators whose work has been cited in the previous pages all kept their products continuously in water or damp air until their strength was tested. Waterproofings may frequently be used in mortar or concrete which may commence to dry out within a very few hours after it attains its initial set. The question now to be discussed is the influence of the initial curing treatment. If a concrete containing a water repellant waterproofing were to be allowed to dry out within a few days after it was poured, and therefore while it was still weak, it would seem that it would gain strength only slowly, even if it were later exposed to moisture, because of the very fact that it was waterproofed. The following experiments by one of us throw some light on this.

Blocks 4 x 4 x 6 in. were cast in duplicate from a mortar of 1 part cement to 3 of sand with sufficient water to make the mixture rather stiffly plastic, and with either lime or aluminum soap incorporated as a waterproofing. The soaps were pure products prepared in the laboratory and incorporated in rather large amount so that the mortar contained from

0.23 to 0.48 per cent fat acid as soap. The sand in some tests was a natural bank sand with fair gradation and in others it was a washed sand. After the blocks were removed from the mould they were weighed and one of the pair was immersed in water and the other left in room air for a specified time. After the initial period, the blocks were alternated in the conditions of storage, and record was kept of the change in weight as an indication of water absorption. Space does not permit a review of all of these absorption tests, but the results may be summarized by the statement that the insoluble soaps all showed definite waterproofing properties. The measurements of water absorption were repeated at somewhat irregular intervals for three years and then the blocks were left undisturbed either in air or water for a number of years. They were finally crushed after

TABLE IV.—EFFECT OF INITIAL CURING CONDITIONS ON COMPRESSIVE STRENGTH OF BLOCKS OF 1: 3 MORTAR WATERPROOFED WITH INSOLUBLE SOAPS. CRUSHED WHEN DRY AT AGE OF APPROXIMATELY 11 YEARS.

Test No.	Waterproofing % by wt. of Cement	Waterproofing % Fat Acid in Mortar	Initial Storage Days Wet	Initial Storage Days Dry	Total Immersion in water	Crushing strength lbs. per sq. in.		Relative Strength of blocks initially dry. Per Cent
159 A	1%A	0.23	21		8 yrs.	2980		
B	"	0.23		31	8 mos.		1260	42.3
C	2%A	0.46	28		8 yrs.	2770		
D	"	0.46		38	8 yrs.		1160	42.0
E	1%B	0.24	28		8 yrs.	4100		
F	"	0.24		38	8 yrs.		3370	82.3
G	2%B	0.48	28		8 mos.	3210		
H	"	0.48		38	8 yrs.		1770	55.2
I	1%B	0.24	28		8 yrs.	3580		
J	"	0.24		38	8 yrs.		1480	41.3
K	2%B	0.48	28		8 mos.	2180		
L	"	0.48		38	8 yrs.		1030	47.3

Waterproofing A was a pure lime soap, Calcium Oleate.
Waterproofing B was pure aluminum soap, Aluminum Stearate.

being thoroughly air-dried at room temperature when they were about eleven years old.

The results of the compressive tests are given in Table IV. The conditions were not comparable enough to enable the various pairs to be compared with each other. Comparison between the members of each pair is, however, proper and shows clearly that even eight years' immersion in water was unable to overcome the handicap caused by the early drying of one of the pair. All but one of these blocks, which were allowed to dry out as soon as removed from the mould, was subsequently immersed in water at various times for a total period of about eight years; yet the average strength of these blocks, dried after removal from the mould, averages only about fifty per cent of the companion blocks which were immersed for the initial period and afterwards subject to alternate dryings and immersions. The evidence clearly points to the necessity of keeping

waterproofed concrete damp until it has had an opportunity to become fairly strong.

As stated above, the lack of uniformity in the conditions surrounding the making and storage of these blocks prevents a direct comparison of the various lots with each other. It is further to be noted that the amount of fat acid in the blocks, 0.23 to 0.48 per cent, is ten times as much as was necessary to give fairly efficient results. Yet the minimum strength of the blocks which were initially cured wet 21 days or longer was 2180 lb. and the average strength was 3130 lb. per sq. in. Even the relatively excessive amounts of soaps used in these tests permitted fairly satisfactory strengths when broken after eleven years.

Table V.—Percentage of Fat Acid in Raw Materials, Mortar and Concrete.

First Series of 1: 2: 4 Concrete Cylinders with Proportions of Waterproofing Specified by Manufacturer.

Designation	Proportions	% fat acid in paste	% fat acid in mixing water	% fat acid in concrete as mixed
A	1: 36 vol. water	19.2	0.42	0.030
B	1: 36 vol. water	22.6	0.59	0.051
C	2% wt. cement	13.7	0.43	0.035

Second Series of 1: 2: 4 Concrete Cylinders and Blocks of 1: 3 Mortar With Double the Proportions of Waterproofings Specified by Manufacturers.

Designation	Proportions	% fat acid in paste	% fat acid in mixing water	% fat acid in concrete as mixed
A	1: 18 vol. water	19.2	0.84	0.076
B	1: 18 vol. water	22.6	1.18	0.109
C	1: 18 vol. water	13.7	0.76	0.070

The tests cited above indicated the importance of initial curing conditions but it seemed desirable to make a new series with amounts of soap more clearly comparable with current practice. In the earlier set the soaps used were prepared in the laboratory but for this series commercial soaps prepared for waterproofing concrete were used.

Tests with Commercial Soaps as Integral Waterproofings.

Three commercial soaps were tested, all furnished in the form of pastes which were added to the mixing water. A and B were aluminum soaps emulsified by ammonia and C was an ammonium soap. In the first series of 1: 2: 4 compression cylinders the soaps were added in the proportions specified by the manufacturers for mass concrete. Twice the amount speci-

fied was used in the second series of 1: 2: 4 compression cylinders and in the 1: 3 mortar used for blocks and expansion bars. The data relating to the proportion of fat acids in the pastes and in the mixed concrete are given in Table V. It will be noted that the amount of fat acid recommended for mass concrete varies from 0.030 to 0.051 per cent of the weight of the concrete when poured.

Tests of Permeability.—Tests of permeability to water under pressure were made on one-foot cubes of 1: 2: 4 concrete made from washed sand and gravel with water containing the amount of soap specified by the manufacturers for mass concrete. Sufficient water was used to give a four-inch slump. Blocks of plain concrete were made in the same manner as those containing soap. Steel inserts made from 3-in. pipe nipples and adapters

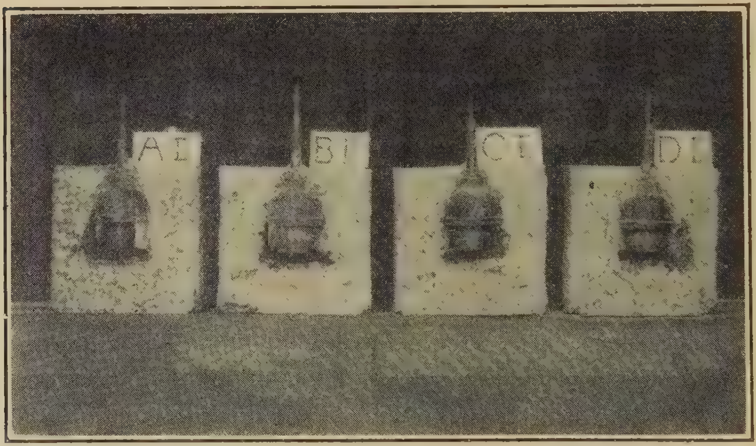

FIG. 1.—APPEARANCE OF CUBES AFTER PERMEABILITY TEST.

were embedded in the concrete as shown in Fig. 1. The large open end of the inserts was closed with a porous disc made of Ottawa sand mortar, mixed in the proportions of 1: 6 by weight and reinforced with iron gauze. This disc was attached to the steel insert by means of neat cement paste. The concrete was placed in the cubic foot forms to a depth of 4½ in. and the steel insert put in and held in place by means of a bracket. The steel insert was then completely covered with approximately ½ in. of neat cement paste to prevent leakage of water along the steel and the form filled with concrete. All specimens were left in the forms for 2 days, then placed in damp sand for 5 days, and then placed in air of the laboratory until tested at the age of 28 days.

Two sets of blocks were tested. The first set started with water at low pressure and the pressure was increased at intervals to 40 lb. per sq. in. No moisture having appeared on the surface of any specimen, the

test was discontinued after 24 hours and the blocks were split open with a hammer and chisel. Visual inspection of the broken surfaces showed that the penetration of water was somewhat deeper in the specimen made without waterproofing. A second set of blocks was then tested under the full city water pressure which fluctuated between 40 and 60 lb. The pressure was maintained for 36 hours and no moisture having appeared on the surface of any specimen, the blocks were then split open as before. Their

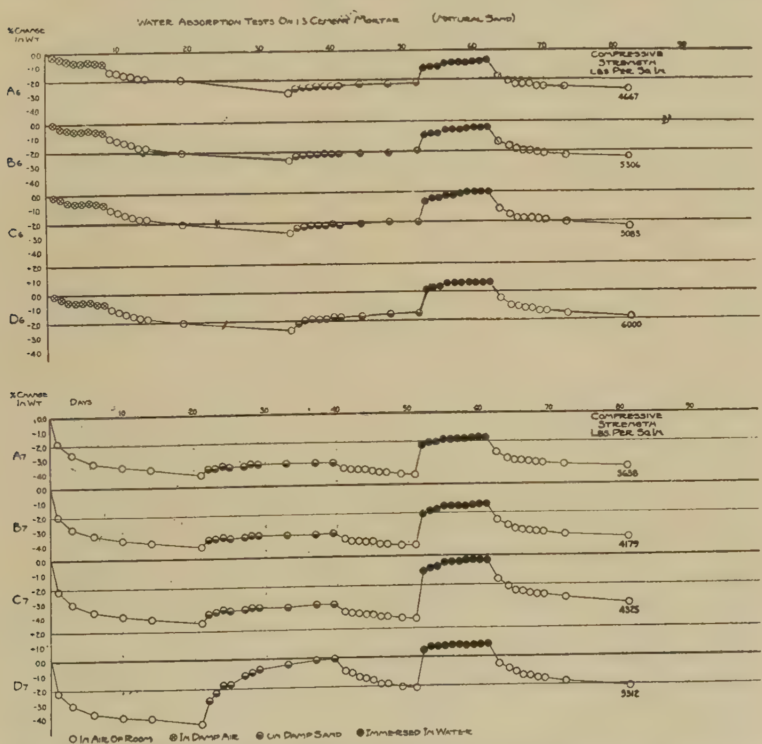

FIG. 2.—WATER ABSORPTION OF BLOCKS OF 1: 3 MORTAR SHOWING INFLUENCE OF SOAPS USED AS INTEGRAL WATERPROOFING.

appearance immediately after splitting is shown in Fig. 1 where *D* is the specimen of plain concrete and *A*, *B*, and *C* are the specimens made up with the soaps bearing the corresponding letter. It is seen that the water had penetrated the plain concrete for a distance of about two inches on one side of the pipe and that it had not penetrated half that far in any of the other specimens. The test is confirmatory evidence of the ability of well-made 1: 2: 4 concrete to resist penetration by water if it is properly cured and if the layer of concrete is sufficiently thick. The colloid swells when

it comes in contact with water and interposes so much resistance that the penetration of water finally ceases. It is also to be noted, however, that even in concrete as carefully made as these test specimens, lack of uniformity was made evident by uneven penetration of the water. The addition of soaps to the mixing water decreased the penetration of the water in every case in this set as well as in the first set which is not pictured here.

Tests of Water Absorption by 1:3 Mortar.—The purpose of these tests was to determine the effects of these soaps on absorption of moisture through capillary action. Rectangular blocks with a base of 4 in. and a height of 6 in. were prepared from a mixture of 1 part of cement by weight to 3 of Ottawa sand. The waterproofings were added in the proportion of 1:18 of the volume of the water, this being twice the quantity recommended for mass concrete. The percentage of fat acid in the mixing water and in the mortar may be obtained from Table V. The water absorptive properties of these blocks were tested by placing the dry blocks on wet sand in a covered box and determining the rate of rise of water through capillary action by visual observation and by the increase in weight. The blocks were removed from the moulds after 24 hours and pairs were separated for treatment. One member of each pair was allowed to stand in the air of the laboratory for 17 to 19 days and the other was kept damp for 7 days before being allowed to become dry. Both sets were then placed on damp sand to test their water absorptive properties, and later dried again and subjected to further treatment. Sixteen blocks or eight pairs were made and tested in this manner from Ottawa sand. Sixteen other blocks were made in a similar manner from natural sand. Lack of space prevents the presentation of these results in full and only the results of four pairs are expressed graphically in Figs. 2 and 3. In both figures the letters *A*, *B*, *C* represent the blocks waterproofed with the soap of the same designations. The letter *D* represents the blocks of plain mortar without waterproofing.

The history of four pairs of these blocks is shown graphically in Fig. 2. One block of each pair was kept damp for the first 8 days after it was cast, while its companion was allowed to dry out in the room air as soon as it was taken from the mould and was left in room air for 21 days. The conditions of later storage were equalized so that each block, while in a dry state, was placed on damp sand and allowed to absorb water for 18 days, was then immersed in water for 10 days, then dried in air for 20 days, and finally crushed when 82 days old. The value of the initial storage of 8 days in damp air is shown in several ways. The rate of absorption of water through capillary action is shown in Fig. 3, both by the curve of change in weight and by a diagrammatical representation of conditions as shown by a visual inspection of the blocks. It will be noted that in the block *D7* of plain mortar, the water rose nearly to the top of the block after standing 7 days on damp sand. This was the block which had been placed in air immediately after removal from the mould. The companion

block $D6$ which was kept damp for the first 8 days of its existence only allowed the water to rise to about one-sixth of its height in the same period, because of the better development of the colloid. The blocks containing the soaps A, B, and C were, however, able to prevent the absorption of moisture quite completely, even in the case of the blocks with initial

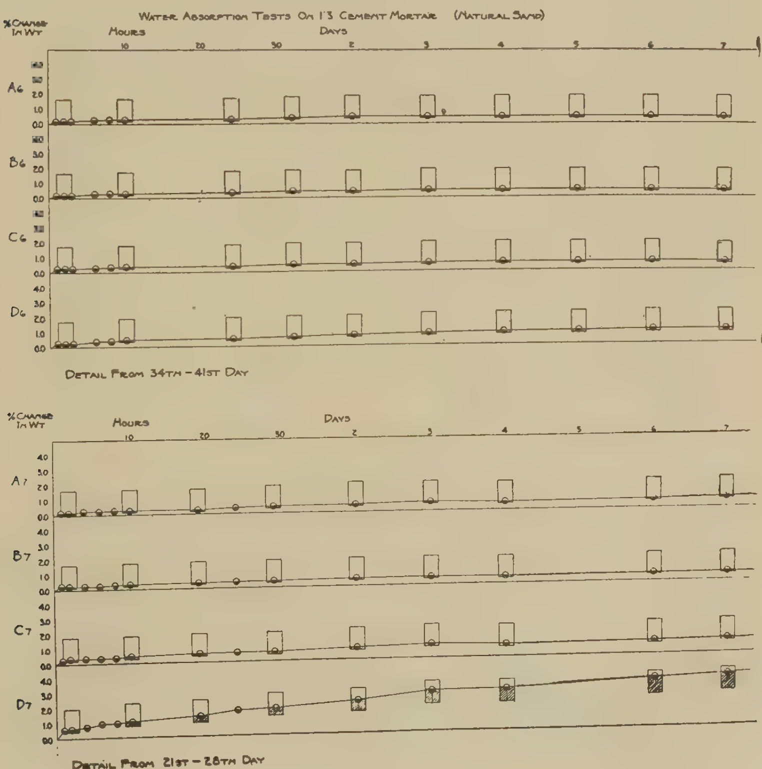

FIG. 3.—DETAIL OF FIG. 2 SHOWING INFLUENCE OF SOAPS USED AS INTEGRAL WATERPROOFING ON ABSORPTION OF WATER BY BLOCKS OF 1 : 3 MORTAR THROUGH CAPILLARITY.

storage in air as is shown by the details of Fig. 3 as well as the curves of Fig. 2.

The value of the waterproofing is shown also on immersion of the blocks. The blocks D6 and D7 without waterproofing absorbed water on immersion so that their weight became about one per cent greater than the initial weight. The blocks C6 and C7 absorbed water to about their initial weight while the blocks A6 and B6 and the blocks A7 and B7 failed to attain their initial weight by from one to two per cent. The value of the

soaps is, therefore, distinctly shown in the case of the blocks which became dry soon after they were made. The soap was undoubtedly effective in the

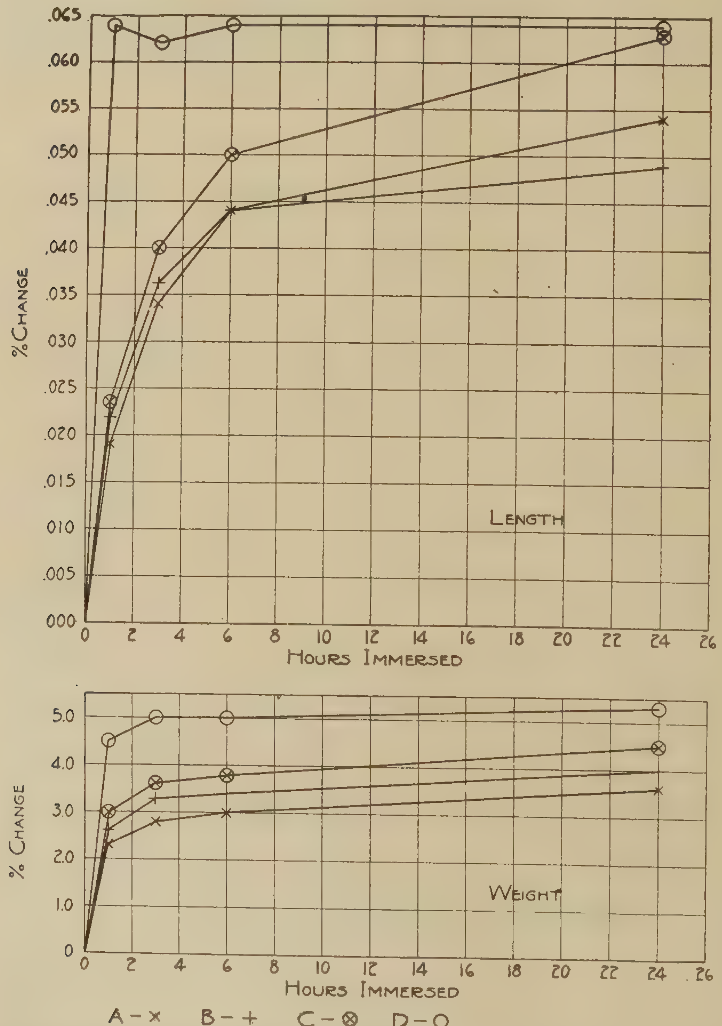

FIG. 4.—VARIATION IN LENGTH AND WEIGHT OF SMALL BARS OF 1 : 3 MORTAR, WITH AND WITHOUT WATERPROOFING WHEN IMMERSED IN WATER.

other blocks which did have their colloid developed by curing in damp air for eight days, but in this latter series the soaps were not called on to prove their value. They acted merely as a line of secondary defense.

The figures for compressive strength of these blocks, all of which were crushed at the age of 82 days, also showed the effect of initial conditions of curing and are given in Table VI. The blocks cured wet for the first 8 days were all stronger than the companion blocks allowed to dry out at once after removal from the mould, and the difference in favor of the blocks cured wet was greater in the case of the waterproofed blocks than in the case of those that were not waterproofed. The blocks containing soaps *A* and *B* which were allowed to become dry as soon as removed from the mould showed only 78 per cent of the strength of the companion blocks cured wet for the first 8 days while the blocks cured dry without water-

TABLE VI.—EFFECT OF INITIAL CURING CONDITIONS ON COMPRESSIVE STRENGTH OF BLOCKS OF 1: 3 MORTAR CRUSHED AT AGE OF 82 DAYS.

	Compressive strength and conditions of initial storage		Relative Compressive strength of dry blocks
Designation	*Damp*	*Dry*	
A	4667	3658	.78
B	5306	4179	.78
C	5083	4325	.86
D	6000	5312	.88

The A, B, and C blocks contained soaps in double the amount recommended by the manufacturer. The D blocks did not contain any soap.

proofing showed 88 per cent of the strength of the blocks cured in the wet state initially. Waterproofing *C* was not as effective as *A* and *B*. The later treatment of the blocks permitted those without waterproofing to gain strength more rapidly because the soaps prevented free access of water to the blocks containing them. This series of blocks is referred to again in the general discussion and the results on the three other sets, each of eight blocks are also discussed in the same place. The effect of soaps on cylinders of concrete cured under identical storage conditions is discussed in a following section.

The Effect of Soaps on Expansion and Contraction of 1: 3 Mortars.

At the same time the blocks were made from 1: 3 mortar as described in the preceding section expansion bars four inches long and one inch square were made from the same mortar for a study of volume changes. Both the length and weight of these bars were observed in an attempt to study the correlation between the two sets of changes. The general conclusion is that the soaps retarded the penetration of water but that if sufficient time was allowed the changes in volume and weight of these small bars became substantially the same whether soaps were incorporated or not. The more porous mortars made with Ottawa sand showed less elongation than the

denser mortars made with well graded natural sand in accordance with the usual experience. The effect of the waterproofings is shown most clearly in the rate of absorption when immersed in water. This is evident in Fig. 4 where the water absorption of a set of four bars is shown. These were placed in water for 6 days after removal from the moulds, then in dry air for 16 days and were then immersed in water with the results plotted in Fig. 4. The bar D without waterproofing gained nearly all of its final increase in weight and length in the first hour whereas the waterproofed bars gained only about half of their final increase in weight and one-third of their increase in length in the same period. The curves of Fig. 4 show the development of the expansion during the first 24 hours of immersion. After 48 hours the increase in length of the various specimens had become substantially equalized. These changes of length with absorption of water are characteristic of all structures made from portland cement and cannot be prevented if the moisture content changes. The only way to prevent volume changes is to keep the moisture constant.

The general result of all these tests on bars of small cross-section, is that waterproofing with an insoluble soap retards the penetration of water on immersion to a marked degree and retards to a still greater degree the expansion which always follows absorption of water. The waterproofing, however, merely retards and does not prevent the absorption of water by these small units. If small bars like these are immersed for a sufficient time, the waterproofed bars will finally expand to practically the same lengths reached by the plain mortar in a shorter time. Blocks of large size which contain soaps as waterproofings will absorb water more slowly and will not permit of such deep penetration of the water. Large blocks which have been waterproofed may therefore show very little change in volume even on long immersion, because the penetration of water has been stopped a short distance below the surface.

Compressive Strength of Cylinders of 1:2:4 Concrete With and Without Soaps as Waterproofings, and Cured under Identical Conditions.

Standard 6 x 12-in. cylinders were made in the proportions of 1:2:4 by volume from the same material used for the large cubes tested for permeability. In the first series soaps were added to the water in the proportions specified by the manufacturers as given in Table V. Cylinders without waterproofing were made for comparison. Water enough to make a 2½-in. slump was used and all of the specimens were cured in the form for 2 days, then removed and placed in damp sand up to the age of 7 days. Those tested at ages greater than 7 days were cured for the remainder of the time in the air of the laboratory. Three cylinders from each lot were broken at a time and the average results are reported in Table VII. It is seen that waterproofing A did depress the compressive strength but that the other two did not affect it within the limits of error of the work.

When it was seen that the quantities of waterproofing recommended by the manufacturers did not have a marked effect on the compressive strength,

a second series of cylinders was fabricated with the same proportions as the first series except that the amount of waterproofing was approximately doubled and somewhat more water was used in the mix so that the slump was about 4 in. Figures showing the amount of waterproofing are given in Table V. The cylinders were kept 2 days in the form, then stored in damp sand for 5 days and the balance of the time in air. Details of compressive strength are given in Table VIII.

It is to be remembered that these cylinders contained about double the amount of soaps recommended by the manufacturers. The compressive strength at 7 days was diminished in every case by the waterproofings but

TABLE VII.—COMPRESSIVE STRENGTH OF FIRST SERIES OF CYLINDERS OF 1:2:4 CONCRETE WITH AND WITHOUT ADDITION OF SOAPS TO THE MIXING WATER.

The soaps were added in the proportions recommended by the manufacturers and the cylinders were kept damp for 7 days and the balance of the time in air.

Waterproofing Age when Tested	Compressive Strength in Pounds per Square Inch A	B	C	None
7 days	1352	1551	1463	1519
28 days	2320	2908	2690	2778
70 days	2957	3093	3132	3197
		Relative Strength		
7 days	89	102	96	100
28 days	84	104	97	100
70 days	92	97	98	100
Average	88	101	97	100

the inferiority disappeared considerably at the later periods. If the results from the four periods, 28 days, 90 days, 1 year, and 2 years are averaged the relative strength for the cylinders waterproofed with B and C is 96 respectively, while A is 81. These results will be analyzed more closely in the following section of this paper.

General Discussion of Effect of Soaps on Strengths.

The blocks of 1:3 mortar which have already been described were crushed when they were 80-83 days old after having been subjected in groups of four to various conditions of storage. The history of two of the groups is shown graphically in Figs. 2 and 3. Each group comprised one block for each of the three waterproofings and one block with no waterproofing. There were two series, one made from Ottawa sand and one from

a good quality of natural sand. The amount of mixing water used was constant for each series, and a record of the changes in weight was kept. The blocks were thoroughly air-dried before crushing and the increase in weight over the weight of the raw materials gives an approximation of the amount of combined water at that time. The compressive strength of the blocks is plotted against this percentage of combined water in Figs. 5 and 6. In these figures as in the others the letter D stands for the plain mortar

TABLE VIII.—COMPRESSIVE STRENGTH OF SECOND SERIES OF CYLINDERS OF 1:2:4 CONCRETE WITH AND WITHOUT ADDITION OF SOAPS TO THE MIXING WATER.

The soaps were added in about double the quantity recommended by the manufacturer and the cylinders were kept damp for 7 days and the balance of the time in air.

Waterproofing Age when Tested	Compressive Strength in Pounds per Square Inch A	B	C	None
7 days	1191	1331	1305	1671
28 days	2114	2320	2578	2585
90 days	2210	2780	2827	2923
1 year	2184	2566	2287	2357
2 years	2240	2293	2277	2577
		Relative Strength		
7 days	71	80	78	100
28 days	82	90	100	100
90 days	76	95	97	100
1 year	93	109	97	100
2 years	87	89	88	100
Average at all ages	81	93	92	100
Average at ages after 7 days	84	96	95	100

with no waterproofing and the letters A, B, and C stand for the respective waterproofings. It will be seen that the crushing strength varies directly with the combined water both in the set with Ottawa sand and that with natural sand. The blocks without waterproofing, marked with the letter D, show the highest strength because of their ability to absorb water rapidly when opportunity offered and therefore resume the hardening process. The blocks containing soaps had lost this ability to absorb water, and, therefore, did not become so strong. These two series afford quantitative

evidence of the need for keeping waterproofed concrete wet until it has attained a reasonable strength. After it has become dried, it will absorb water very slowly and hence gain strength very slowly.

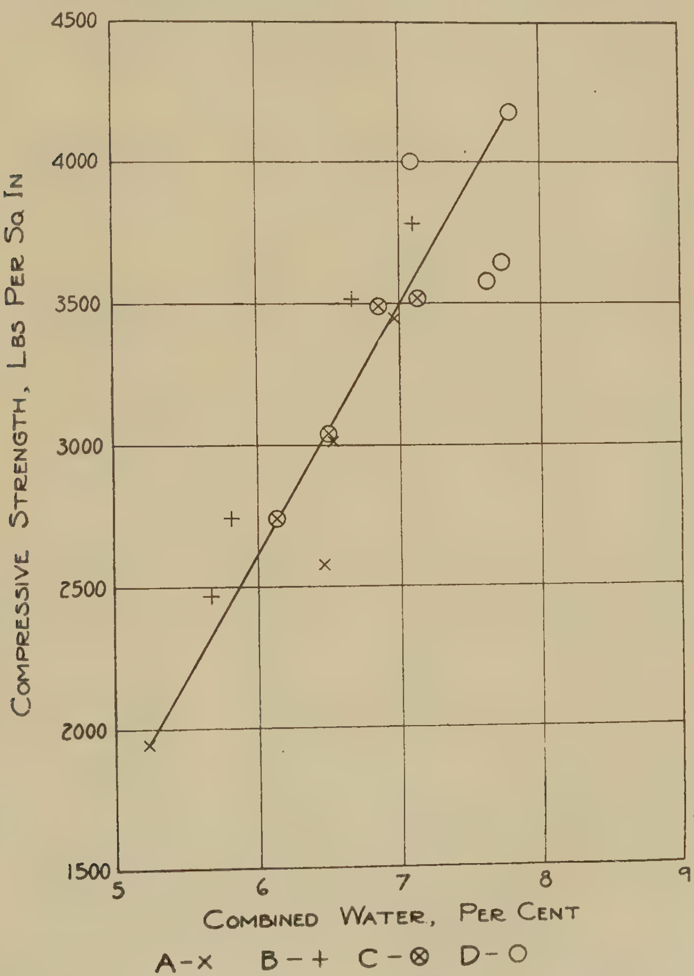

FIG. 5.—COMPRESSIVE STRENGTH OF 1 : 3 MORTAR BLOCKS, MADE WITH OTTAWA SAND, WITH AND WITHOUT WATERPROOFING, AS A FUNCTION OF THEIR PERCENTAGE OF COMBINED WATER.

The study of the compressive strength of the cylinders of 1 : 2 : 4 concrete which has been previously reported showed that the soaps B and C had practically no harmful effect on the strength of the concrete when used

in the proportions recommended by the manufacturers and only a slight effect for ages beyond seven days when twice the recommended amount was

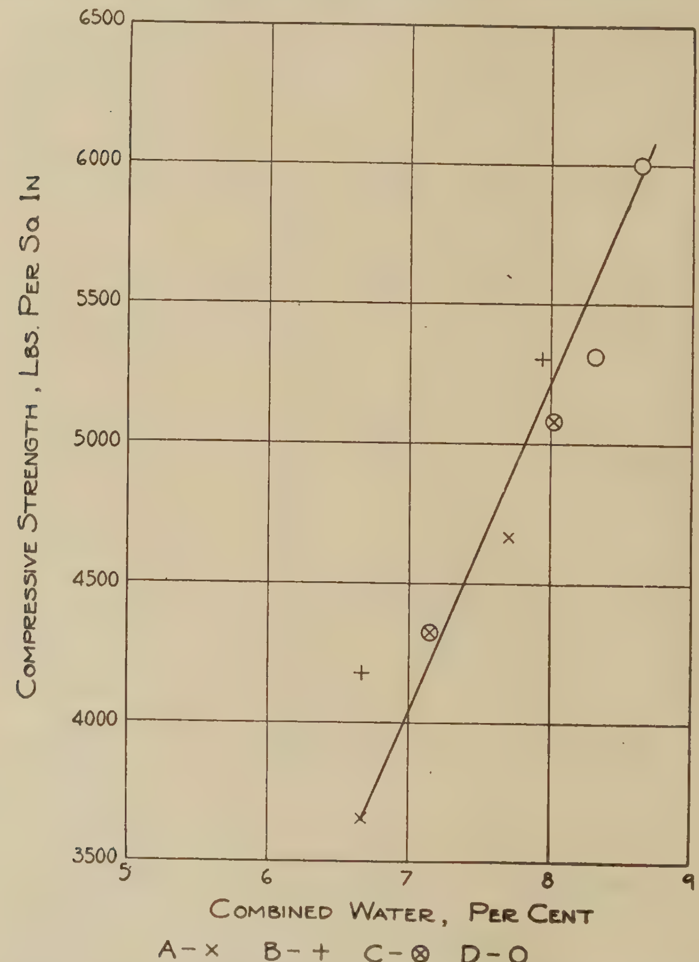

FIG. 6.—COMPRESSIVE STRENGTH OF 1 : 3 MORTAR BLOCKS, MADE WITH GRADED SAND WITH AND WITHOUT WATERPROOFING AS A FUNCTION OF THEIR PERCENTAGE OF COMBINED WATER.

used. In both of these series of cylinders, soap A did show a harmful effect on the strength. The members of both of these series had been stored under identical conditions. The chemical composition of A did not differ

much from B and did not give any apparent explanation of the discrepancy. Fortunately a careful record had been kept of the weights of these cylinders, and the weight at time of testing is plotted against compressive strength of the first series of cylinders in Fig. 7. Each point represents the average weight and strength of three cylinders and is therefore fairly rep-

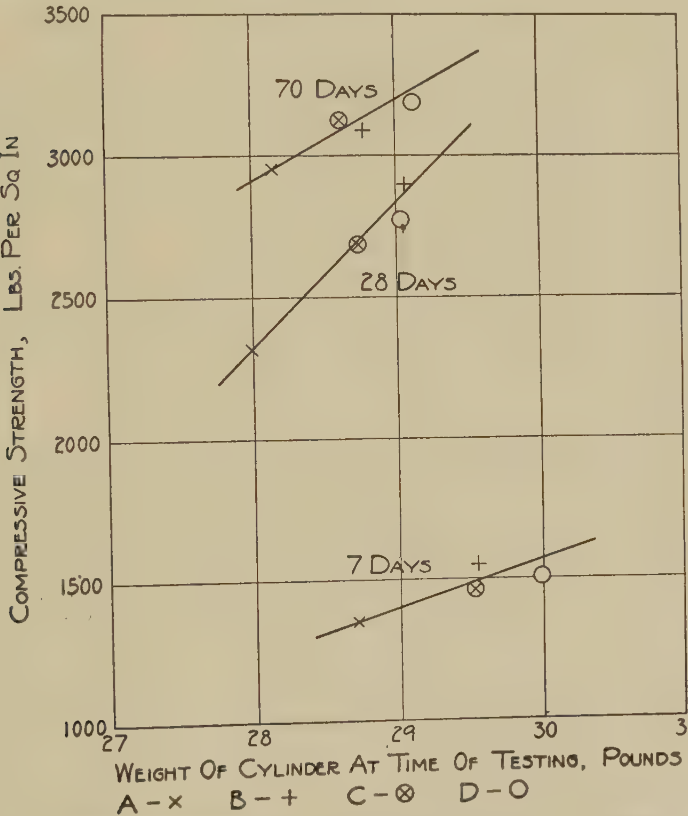

FIG. 7.—COMPRESSIVE STRENGTH OF CYLINDERS OF 1: 2: 4 CONCRETE, WITH AND WITHOUT WATERPROOFING, AS A FUNCTION OF THEIR WEIGHT WHEN TESTED.

resentative. It will be seen that the cylinders made with the A waterproofing show the lowest weight as well as the lowest compressive strength in every case. The second series of cylinders made with natural sand confirms this result, but is omitted from this paper because of the limited space. The explanation of the low strength with waterproofing A lies in its tendency to foam during the mixing process thus entraining air in the

concrete and making it weaker. The other waterproofings did not show this foaming tendency and therefore did not cause any material weakening of the concrete. The same behavior is to be noted in the blocks of 1: 3 mortar. All of the eight blocks waterproofed with A have the lowest compressive strength, and all but one of them the lowest weight of their respective series.

Conclusions.

Soaps added in the concrete mixer are effective in preventing the absorption of water by capillary action, even when added in very small amounts if they are added in such a form as to ensure extremely fine subdivision and dispersion in the finished concrete. As little as 0.05 per cent of fat acid in the form of soap based on the weight of concrete is sufficient if it is finely divided and uniformly distributed, and if the concrete is well made and not cracked. Soaps will not in themselves entirely prevent the penetration of water under even rather slight pressure but they will be of assistance. If the concrete is well made and thoroughly cured, the colloid near the surface swells when it becomes wet and fills the pores so completely that the penetration of water may stop before the water has progressed many inches from the surface, even if the concrete is not waterproofed.

This ability of plain concrete to resist water comes only with time and the development of sufficient colloid to fill the pores. Concrete which becomes dry within a few days after it is made does not possess sufficient development of the colloid to prevent the absorption of water. Soaps are as effective in fresh concrete as in that which has aged and assist in preventing the penetration of water even under pressure. Insofar as the concrete becomes wet it expands, and it contracts when it becomes dry, irrespective of the presence of soaps, which are effective only in preventing access of water.

The strength of concrete is not impaired by the addition of soaps in the proportions stated above, provided it is kept damp continuously after pouring until it has attained the desired strength and provided the soaps have not caused foaming and consequent entrainment of air. The strength of concrete of a given composition is a function of the amount of cement which has reacted with water and may be plotted as a function of the water held in combination. Plain concrete which has been allowed to dry and subsequently becomes wet will resume the hardening process and gain in strength. Concrete containing soaps gains strength very slowly after it has once become dried because it resists the absorption of water. Special pains must, therefore, be taken to keep concrete containing soaps damp for at least seven days if it is to attain a strength comparable with plain concrete.

The shrinkage of concrete on drying is a function of the quantity of cement and the degree of its hydration and will not be lessened by the use of soaps. The same precautions must therefore be taken to prevent shrink-

age cracks in waterproofed as in plain concrete. Soaps are not able to prevent the penetration of water through cracked concrete. They will retard the penetration of water into dry concrete, and therefore will be helpful in preventing volume changes due to alternate exposure to wet and dry conditions in structures such as stuccos and pavements. If the concrete is kept damp, as all concrete should be, until it is properly cured, there should be only slight diminution in strength due to the waterproofing. Conditions may readily arise in service where the waterproofed concrete will be distinctly stronger than the plain concrete. A concrete slab wet and expanded on one side and dry and contracted on the other side is subject to powerful shearing stresses which weaken it materially. A slab which is waterproofed should not absorb water to nearly the same extent and therefore should not be weakened so much under similar adverse conditions.

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Correlated Considerations in Design and Construction of Concrete Bridges.

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Synopsis.

It is the purpose of this paper to emphasize the importance of correlating problems in design and construction of concrete bridges. The term design should imply, not only an acquired aptitude for determing theoretical shapes, but a somewhat instinctive understanding of the problem in construction. By correlating these interdependent considerations, the full meaning of the term design is realized, and results obtained are in direct proportion.

The possibilities in the development of concrete of obtaining effectual economic and architectural values in bridge work are unlimited. Structures may be molded to fit the particular peculiarities of the site with a fine balance of these controlling values. Each problem permits a characteristic solution applicable to the wide variables as they are found in topographical and in a multitude of other differences of the site and to contiguous conditions. So-called "standard designs" are precluded. Any treatise, therefore, attempting to establish fixed relations between design and construction features would be a futile effort. The fact remains that every bridge project in concrete has this problem to meet.

If the structure is to be built in a highly congested district, there is the problem of reducing to a minimum, first, the danger and then the inconvenience to the traveling public; if over a stream of a particularly turbulent nature, there are the extreme hazards of construction in flood time. A reduction of these hazards is the culminating aim of good design and should be made, if necessary, at a sacrifice of seeming economies in theoretical design. The solution calls for the proper determination of construction joints and the stability of the structure therewith; it should be possible to subdivide the work to give continuous uniform operations with a constant minimum labor force. Coincident with the fixing of construction joints are the related considerations of drainage, waterproofing and expansion joints.

To emphasize these important factors, the writer will describe the manner in which they were adjusted in various bridge projects with which he has been identified.

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INTRODUCTION.

The adoption of concrete for use in bridge construction implies, among other aims, an endeavor to secure architectural values. The first impression formed of a completed structure concerns these values. There seems to be a more stringent requirement of structural perfection, in the appearance at least, of a concrete structure than on one built of other material. This holds true not only for bridges, but for all other structures as well. A slight irregularity may develop at a construction joint where the work was stopped for the day due to a spring in the forms, or efflorescence may develop there. These are considered serious faults, but little is said of similar faults even in an exaggerated condition, as they occur in structures built of stone or brick.

Of greater consequence are the leaks that may develop at construction and expansion joints in floor systems or parapet walls of bridges due to improper design of the waterproofing. Surface disintegration starts with the alternate thawing and freezing of the percolating water. These joints necessarily are placed adjacent to a highly ornamental feature of the bridge, usually a monumental pilaster surmounting the face of a pier, and the slightest disfiguration at this point is most noticeable. The prevention of these disfigurations, which in time may reach the magnitude of a failure, are not field problems. They are essentially problems in design. The structural and architectural parts cannot be proportioned without due consideration to these features. The waterproofing problem, and this implies the use of a membrane system, cannot be solved properly after all the structural members have been proportioned.

Why waterproof concrete bridges? Here is a second motive for adopting concrete—the endeavor to obtain permanence. The word permanence as applied to the life of structures carries with it a relative meaning. As generally used, it has an exaggerated meaning. The word longevity better expresses this so-called motive of permanence.

Experience with concrete covers a comparatively short period. Results, however, in the past generation have been so satisfactory as to warrant certain conclusions with regard to the life of concrete. The cause of many failures of earlier experience has been traced to impurities or unsoundness of all the elements, or to constituent parts of the concrete. Tests have been prescribed that preclude such failures. Other investigations and developments have been made that guarantee a fairly definite strength by the proper proportioning of the aggregate and by regulating the cement-water ratio. These regulations of the art guarantee within reasonable limits, a safe and sound material of indefinite life. Referring to structures of appreciable size, it might be said that waterproofing would be unnecessary—if the structure could be built in continuous operation, precluding construction joints; if there were no shrinkage in the setting of the concrete that causes incipient cracking; if expansion joints were unnecessary. These joints and cracks are the vulnerable spots and need protection against the disintegrating effect of the water draining off a structure.

Allied with these considerations is a further precaution in providing ample drainage facilities to carry off the water.

A third motive, adaptation, relates to the potentiality of concrete as it may be adjusted and fitted to the irregularities of the site or the many different geological conditions that may be encountered. The salient characteristics of concrete in its plastic state, which permits molding the structure to fit the site, has revolutionized bridge construction and has made possible great engineering projects. There are so many features obscured to the observer in reviewing a structure as they deal with the measure of adaptation. Many prevailing factors are hidden behind walls or buried under ground. The finished structure never reveals the many important considerations that controlled the method of construction and the hazards accompanying their eventual consummation. They may be of different magnitude for the same final result, depending upon the features that may be considered under the motive of adaptation. This factor alone has more influence in determining the cost and the successful completion of the structure than any other factor or combination of factors.

ARCHITECTURE.

There is no intent here to discuss architectural values in concrete bridge design from the standpoint of correct orders or technique of composition. However, there is required of the engineer a well grounded appreciation of fundamental architectural composition because of the many factors in the preliminary determination of the structural skeleton of the concrete bridge.

General balance in these determinations is a function of the engineer. A definite relation exists between adaptation, on the one hand, heretofore classified as the molding of the structure to fit the peculiarities of the site, and architectural values on the other hand. In the case of an arch bridge, the superimposed loading of the floor system and the relation of the length of span to the height of the structure, affects the shape of the main arch member. The arch can be bent and shaped, so to speak, to adjust its neutral axis to the curve of loading and give approximately uniform pressures throughout the width and length of the arch ring. When these considerations are effectually determined, graceful general composition and maximum economy ensue. The decided impression of beauty then follows in logical sequence, the amount of which depends upon the necessities of the occasion.

With the molding of the architectural composition comes the deliberation relative to minimizing the work and difficulty, first, in the building of the forms, and finally, in their removal. The motive of longevity, to preserve these impressions of beauty, is then in order. This will be the extent of the consideration given to the subject of architecture.

A camber in the floor system may be used to good advantage in gaining additional headroom or vertical clearance for an overhead highway bridge at railroad crossing or additional waterway for structure over

a stream. The grades forming the camber ascending from either end of the bridge have a fine purpose in accelerating drainage of the roadway surface. In addition the camber has an architectural value. Given a level floor, coping and balustrade, their horizontal line in elevation has the appearance of sagging at the crown or center of the main arch. The camber nullifies this optical illusion.

The balustrade, an important but comparatively small feature in decorative art, is taken as an example to illustrate considerations in form design. The question usually arises in the choice between one cast-in-place or the alternative, built of precast sections. Preference given to the cast-in-place type seems justified in consideration of greater esthetic possibilities, and from the standpoint of strength and protection. In these days of intensive automotive transportation, a substantial guard rail is a prerequisite structural component which combined with a high curb affords a safeguard for unmanageable vehicles tending to plunge over the parapet.

Esthetic considerations in design of balustrades generally require sharp edges for projecting members of small dimension. Much of the character of the railing is lost by placing excessive corner moldings in the forms to chamfer these edges. The sharp edge can be obtained, but in so doing extreme care is necessary in perfecting the forms and in puddling concrete while it is being poured. Tamping the outside of the forms with a wooden mallet helps materially to secure good results. But this result cannot be obtained if the forms are not easily removed. To this purpose the angle formed by any projecting or offset member should be made slightly greater than a right angle. Where the projecting member is supported by a curved surface, the interior angle of the curve should be less than 90 deg. These considerations were given in design of the balustrade, a section of which is shown in Fig. 1. There was very little fracture to the sharp edges when the side forms were removed and these fractures were satisfactorily repaired in rubbing of concrete immediately after the removal of forms.

The top surface of the rail member is shaped to the form of a curve. Either the curved surface or an apexed surface formed by inclined planes could properly have been chosen. One important reason, an aspect of workmanship, may be advanced for the preference given to the curved surface; that the inevitable irregularities of finish are not so noticeable. The troweling of the apexed surface may be practically perfect, but slight subsequent shrinkage or settlement of the concrete in the forms has a tendency to twist or distort the line of apex in the most conspicuous manner.

The openings of the balustrade are formed by steep beveled sides, apexed at the center of the railing, which facilitate the withdrawal of their form blocks, arranged in pairs. Placed together, the form blocks are bolted to one of the side form panels, the bolt heads being countersunk in the block to give close bearing of the block with the other side form panel. The blocks are shown in position bolted to side form panel in one of the photographs of Fig. 1. The blocks are readily adjusted to a different grade of the balustrade by drilling another set of holes in the

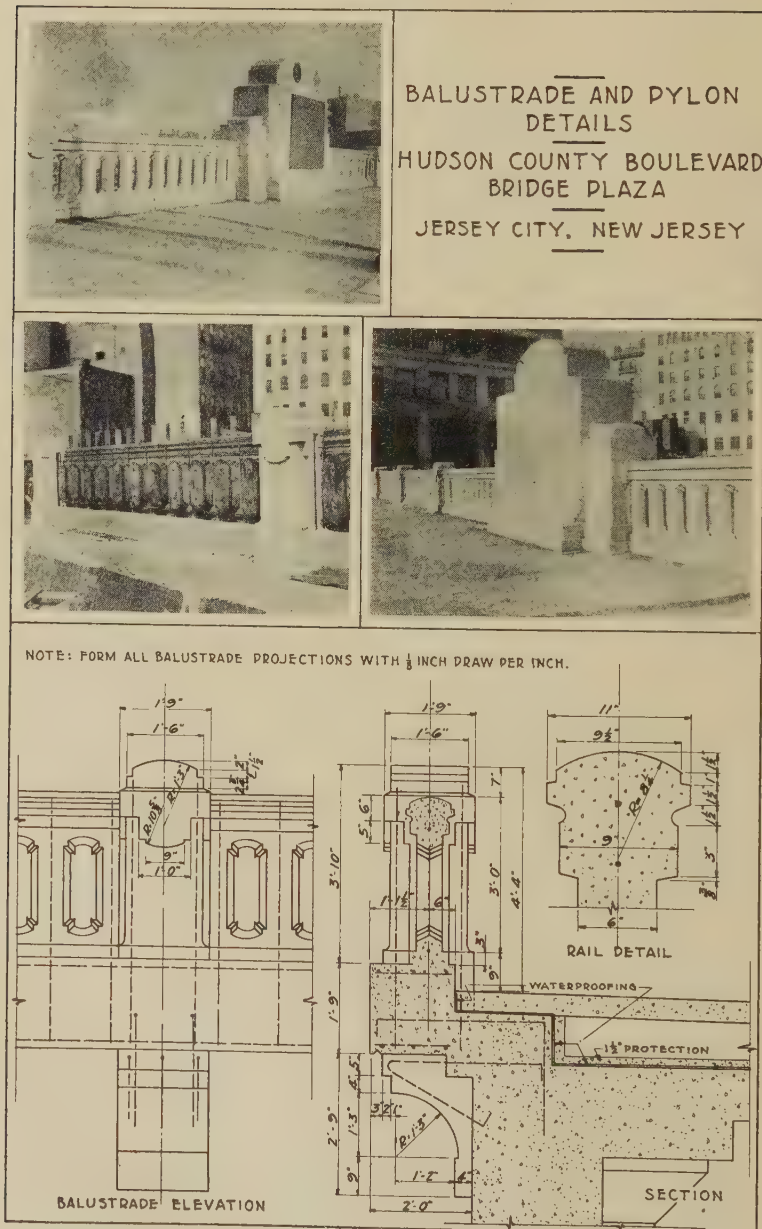


FIG. 1.—HUDSON COUNTY BOULEVARD BRIDGE BALUSTRADE.

side panel. The pair of bolts through each set of blocks holds them substantially in place during the ramming of the concrete. Another set of bolts at greater intervals pierce them and the side form panels in holding the entire unit together.

These main tie bolts are pulled, and both side form panels removed by rotating slightly about their base, 48 hours after the forms are filled. The bolt nuts holding the blocks forming the openings in the balustrade to one side form panel are removed first and when the side panel is pulled off, the blocks remain in the concrete. After the concrete has seasoned four or five days, the blocks are loosened merely by tapping.

Each balustrade panel or section between posts is a monolithic unit sustantially reinforced and held in place by vertical bars in each baluster and horizontal bars through the rail and base. These horizontal bars are not continuous through the balustrade posts. A shallow reglet is formed in each post to hide the construction joint between panel and post and the reglets are lined with two-ply of tar paper to permit minute increments of movement due to temperature changes.

The main parapet coping as shown in drawing, section of balustrade, Fig. 1, is carried 6 in. above the sidewalk level forming a dike to prevent the water of melting snow or driving rains from reaching the outer surface of parapet or facia walls of the bridge, which would take place through the construction joint between coping and balustrade if the top of the coping were placed on a level with the sidewalk. In this design the main floor slab of the bridge is carried through to the parapet, the sidewalk and curb being built after the waterproofing membrane and its 1½ in. of concrete protection has been laid. The waterproofing membrane is brought to a good termination in construction joint between the sidewalk and the coping.

The proper termination of the waterproofing membrane requires considerable study as there is always the danger of water working its way behind and under the membrane. By bringing the waterproofing up the inside of the coping and clamping it down by the sidewalk slab, there is little danger of such a failure. A failure is also minimized by locating the termination at a point of least water accumulation.

A "V" molding is placed on the underside of the coping to form a drip for water falling on the outer surface of the railing and coping and thus preventing this water from following along the underside of the coping to the parapet walls and causing ugly disfiguration by streaking it with dirt accumulations carried with it from the coping and balustrade surfaces.

Referring again to the balustrade illustration of Fig. 1, it is evident that no attempt was made to hide the main expansion joint of the floor system as it is carried vertically through the balustrade. The openings of the balustrade are so proportioned that the center of one is in the vertical plane of the expansion joint giving least amount of detraction architecturally.

Expansion joints in floor systems of bridges and in facia walls of other concrete structures fix salient characteristics in structural design. They are located in a particular position as best fitted to take up movement due to temperature changes, their position and number depending upon the size and configuration of the entire structure. Serious defects are possible if irrevocable architectural composition has been conceived before these essential provisions have been made; if their importance is not fully comprehended or is under-rated or they are neglected in favor of seeming economies in design based on continuity of spans or otherwise. These defects appear in the form of irregular cracks occurring indiscriminately to deface and eventually to impair the strength of the structure. In some structures, expansion joints may be hidden or composed within architectural features so as to hide them from view if considered unsightly. The method of concealing the joint must be simple and direct. Sliding surfaces in contact invariably miscarry. It will be found that almost every effort to fully conceal these joints presupposes unusual care in construction.

The subject could be carried on at great length. Examples could be shown of set back planes at construction joints to hide what would otherwise result in uneven, unsightly joints. In structures built in outlying districts where small plant and small daily operations are imperative, or in large operations such as piers where many daily operations are necessary to complete the shaft, there are the construction joints resulting from these multiple operations. They can be effectually hidden by horizontal rustications form by V-shaped moldings nailed to the inside of the forms. Each operation is stopped at the apex of the "V," thus concealing the construction joint and giving pleasing architectural effects.

The requirement that plans of the structures contemplate these joints is further influenced by details controlling the lengths of the reinforcing steel. With the position of the construction joint fixed the length of the reinforcement is also fixed so that only the lap required to develop adequate bond protrudes beyond the joint. Long dangling rods at these joints seriously handicap the construction by imposing difficulties in the holding of these rods in position and in pouring concrete between them.

There is a broad field for ingenious effort in correlating structural and architectural design to fulfill the strict requirement of structural perfection attached to concrete work, and to preserve this perfection in perpetuity after it has been wrought. A most important avenue of preservation deals with the prevention of surface disintegration resulting from water accumulating, then percolating through expansion and construction joints which will be discussed subsequently under the interdependent subjects of expansion joints, waterproofing, and drainage.

ADAPTATION.

The motive, "adaptation," may be resolved into three formational considerations as the concrete bridge design is adjusted to the irregularities of the site. They deal with geological, lineal and conditional irregularities.

Ascribe principally to geological irregularities their effect on the design of the foundations or footing plan; to lineal irregularities, the effect on the style or type of superstructure due to variations of terrain, limitations of lateral and vertical clearances and to the angle of crossing; to conditional irregularities ascribe the effect on the entire design, of limitations in construction including the fury of the elements—principally flood conditions—possible plant layout, and traffic maintenance requirements in congested districts—pedestrian, vehicular and railway.

The limitations included in geological and lineal considerations are existent and interdependent in every project and are confined alone only to small structures in outlying districts. The adjustment is a matter of balance in design and theoretical considerations dominate.

Add the third consideration, the conditional irregularities, to the problem and the question of method of construction becomes the predominating consideration. The question involves a desirable division of the work into constant workable daily operations. Construction operations carried on by artificial light and overtime effort of single shift of workmen should be prevented if at all possible. It also involves the consideration of minimizing the hazards during construction, a factor largely controlling the cost and the measure of success of the work. Even a fairly remote possibility of a failure causes disorganizing and trying predicaments tending to upset the aim and progress of construction program based generally on fortunate rather than the unfortunate breaks. Above all the safety of life and limb is involved.

Type Selection.

This subdivision of interest fully considered would lead to infinite ramifications which is deemed unnecessary to the ultimate purpose of this paper. Among the series of plates prepared to illustrate the theme, there are those which outline rudimentary shapes. It is possible to take these shapes combined with counterfort, cantilever, and other fundamental, long standing structural ideas reproduced in concrete and build up a structure symbolizing the aspired motives previously outlined.

Attention is called to Fig. 2 which contains the following rudimentary shapes as applied to railroad loading. A in Fig. 2 represents a section of an arch invert culvert built on earth foundation and carrying load of deep fill. On railroad relocation this type was built under fills of 100 ft. in depth. The arch opening varied from 3- to 8-ft. spans. The principal consideration in design was based on the assumption that the weight of the entire fill over the structure is carried by it and transferred uniformly to the base of the invert which is designed as a simple slab to resist the resulting uniformly distributed upward pressure over span of arch opening.

B shows section of reinforced box culvert with railroad loading carried to the slab top by minimum ballast requirement. This type is applicable to spans of 8 to 12 ft. and greater if necessary. Usually, however, if greater spans are required and foundation bearing will permit, the rein-

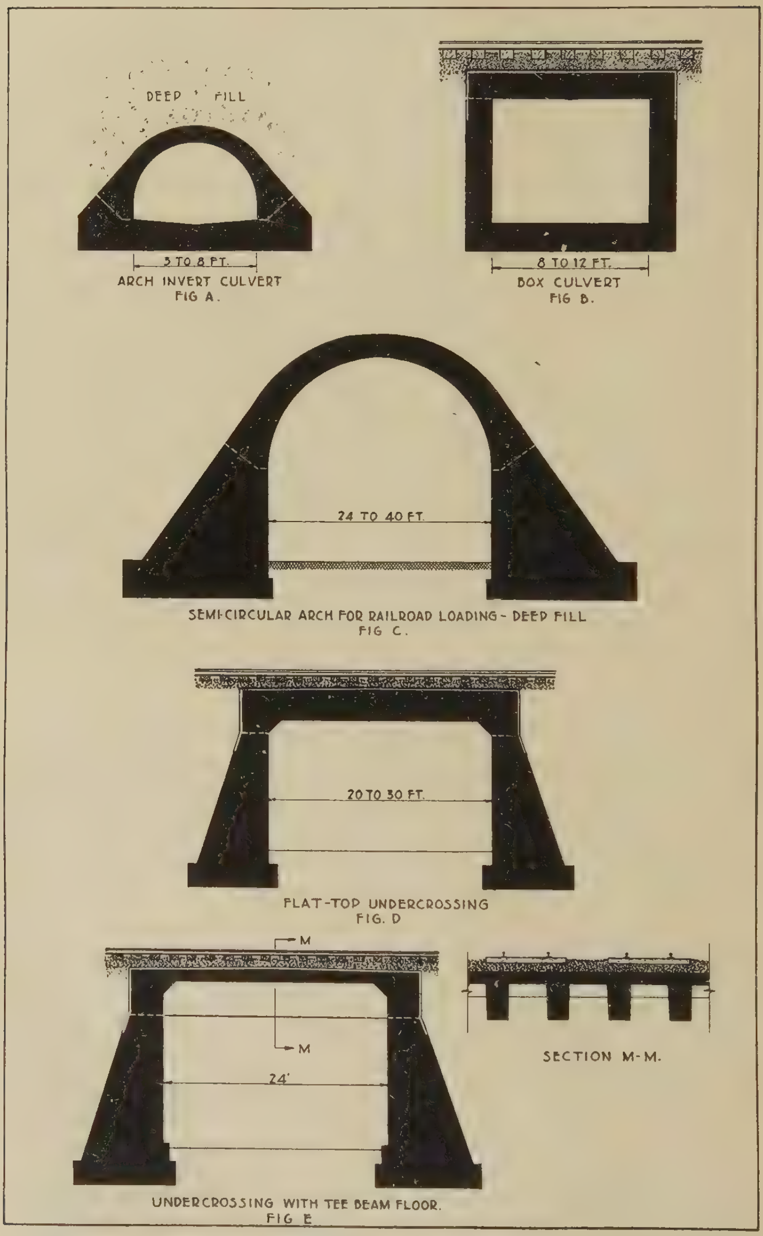


FIG. 2.—RUDIMENTARY SHAPES AS APPLIED TO RAILROAD LOADING.

forced side walls and invert are replaced by gravity abutments as shown by D where span of 20 to 30 ft. is indicated. The reinforced box type is used where scouring action of the stream is so pronounced that tendency prevails to undermine the abutment unless carried to unusual depth or protected otherwise.

The box type is used also for pedestrian subways. The slab top is waterproofed by two-ply asphalt—saturated cotton cloth laid in three swabbings of hot asphalt resulting in a water repellent membrane or blanket about 3/16 in. in thickness. The waterproofing membrane is carried below the construction joint marking the slab seat and in the case of subways it is carried around the entire section if soil is highly saturated with water and it is necessary that dampness be excluded from interior walls. Either a layer of common brick, 1½ in. of asphaltic mastic, or 2 in. of concrete is laid over the membrane as a protection against possible abrasion due to the sliding of ballast or fill, and to further preserve it from the disintegrating effect of varying atmospheric conditions.

A section of a semi-circular arch span 24 to 40 ft. is shown in C. This type has been used under deep fills and elsewhere as clearances permit supported on earth, rock and pile foundations and set on crossing angles of extreme skew over highways and streams. Considering the size of abutments, the semi-circular arch gives better balanced structure for earth and railroad loadings than the arch of segmental, elliptical or other flat shapes. The assumptions used in spread of live-load through fills of appreciable depth give very light loading effect for weight of rolling stock. A wheel or axle loading is assumed spreading on a ½ to 1 slope in direction of track and multiplied for over-lapping of loads of adjacent wheels the amount depending upon the depth of the fill. This uniform longitudinal loading is again spread in the transverse direction over area bounded by same slope lines, ½ to 1, from the edge of ties and lapping loaded area due to loading on adjacent tracks is again considered. For flat top undercrossings of types shown in B and D, the live-load represented by wheel concentration is equated to a uniform loading and spread over a 13-ft. width of slab, the distance center to center of tracks.

A slight variation for the flat top undercrossing is shown in E when greater vertical clearance prevailed prompting tee-beam floor to reduce dead-load weight of fill and slab. Here the live-load of each track is carried by two beams.

A combination of the box culvert and the semi-circular highway arch is shown by sections A and B of Fig. 3. This combination effects considerable saving where stream and roadway are in close proximity and structures are required for both under deep fills. As a matter of fact the stream can be diverted a considerable distance as a result of the saving in cost of its structure. One-half of Section A contemplates earth foundation and light fill and the other half rock foundation at appreciable depth and deep fill. For the latter case an additional amount of reinforcement placed in the roadway slabs counterbalances the horizontal component

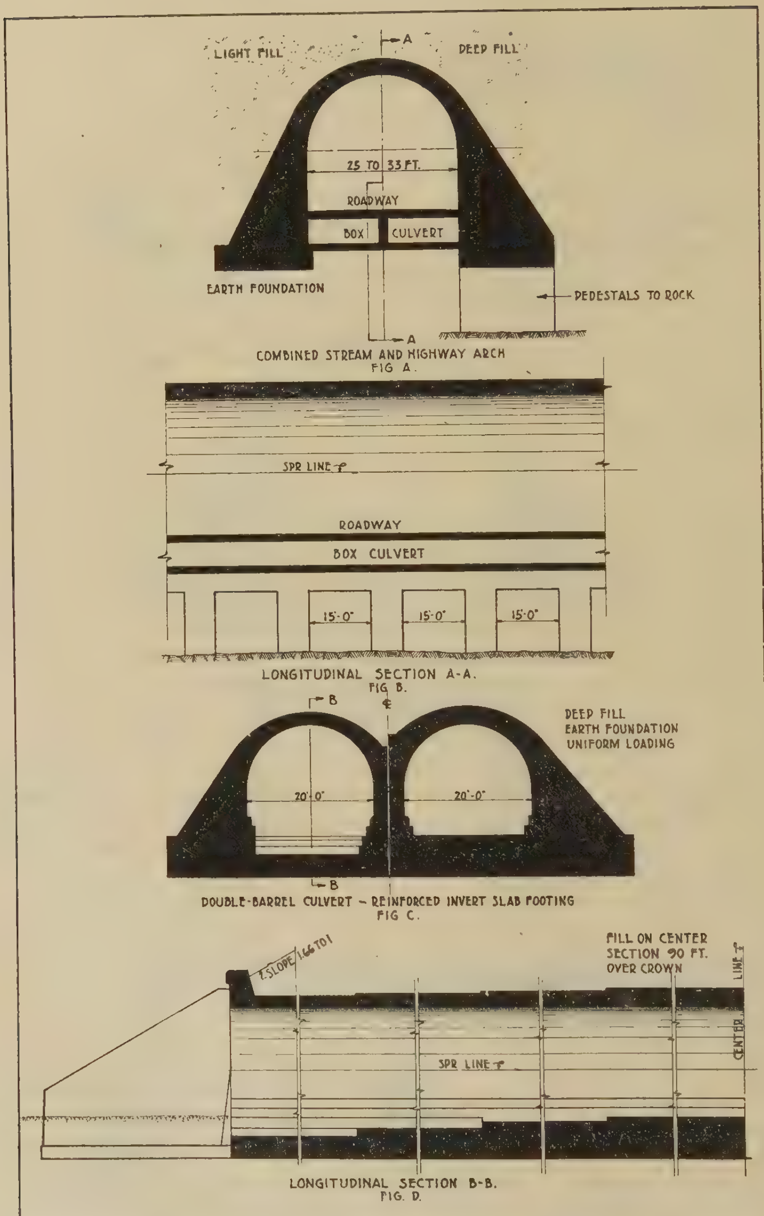


FIG. 3.—COMBINATION OF BOX CULVERT AND SEMICIRCULAR HIGHWAY ARCH.

of the arch thrust at this point making it possible to reduce the cost of foundations to rock by substituting pedestals for solid gravity abutment, which without the counterbalancing would continue to increase in width to rock.

FOUNDATIONS.

The specific geological iregularity which prompted the design of a double-barrel culvert with reinforced invert slab footings as illustrated by Sections C and D of Fig. 4, a large edition of the small semi-circular culvert as shown in A, Fig. 1, was as follows:

Soundings at the site of the proposed culvert along relocation of railroad, showed 8 to 10 ft. of loam and vegetable matter which was a 30-ft. stratum of blue gravel, with varying amounts of fine and sand and clay. At intervals, large boulders were encountered, making the driving of piles difficult. The high water mark and areas provided for by existing structures along the river showed that 360 sq. ft. of waterway were necessary. A 40-ft. semicircular arch was proposed but under the heavy 110-ft. fill the maximum pressure on foundations was found to be far in excess of the safe carrying capacity of the soil. This being impracticable, the alternative was a 20-ft. twin arch. Even with this construction very large abutments would be necessary to keep the maximum toe pressure under six tons per square foot. Further investigation showed a heavily reinforced invert foundation to be not only more stable, but less expensive. By reason of this design the base of the foundation could be placed just below the surface of the gravel, with no possibility of the water undermining the structure, which might be expected if the abutments and pier were built independently of one another. Otherwise the foundations would have to be built at least 4 ft. deeper. The width of the abutments with the invert slab was reduced considerably and the amount placed in the invert span was less than the additional concrete that would be necessary to make the abutments without the invert wide enough for safe bearing and sufficiently deep to insure against scour.

The center section was designed for a maximum fill of 90 ft. over the crown of the arch plus the light effect of engine loading distributed through the deep fill. The ring, abutment, pier and invert were proportioned for this one condition of loading. The line of pressure was everywhere within the middle third section, therefore no tension existed in the concrete. The length of the center section, 40 ft. on either side of the center line, was determined by the distribution of the live-load, which was assumed as spreading through the fill at ½ to 1 slope from the ends of the ties. The remainder of the barrel was divided into three sections of about the same length, 40 ft., each section being designed for the maximum vertical fill over it. The total length of the structure face to face of the parapets is 309 ft. The fill over the structure was made on the usual 1½ to 1 slope, but the length of the barrel was determined by the flatter slope of 1 2/3 to 1 to allow for sloughing of the high fill.

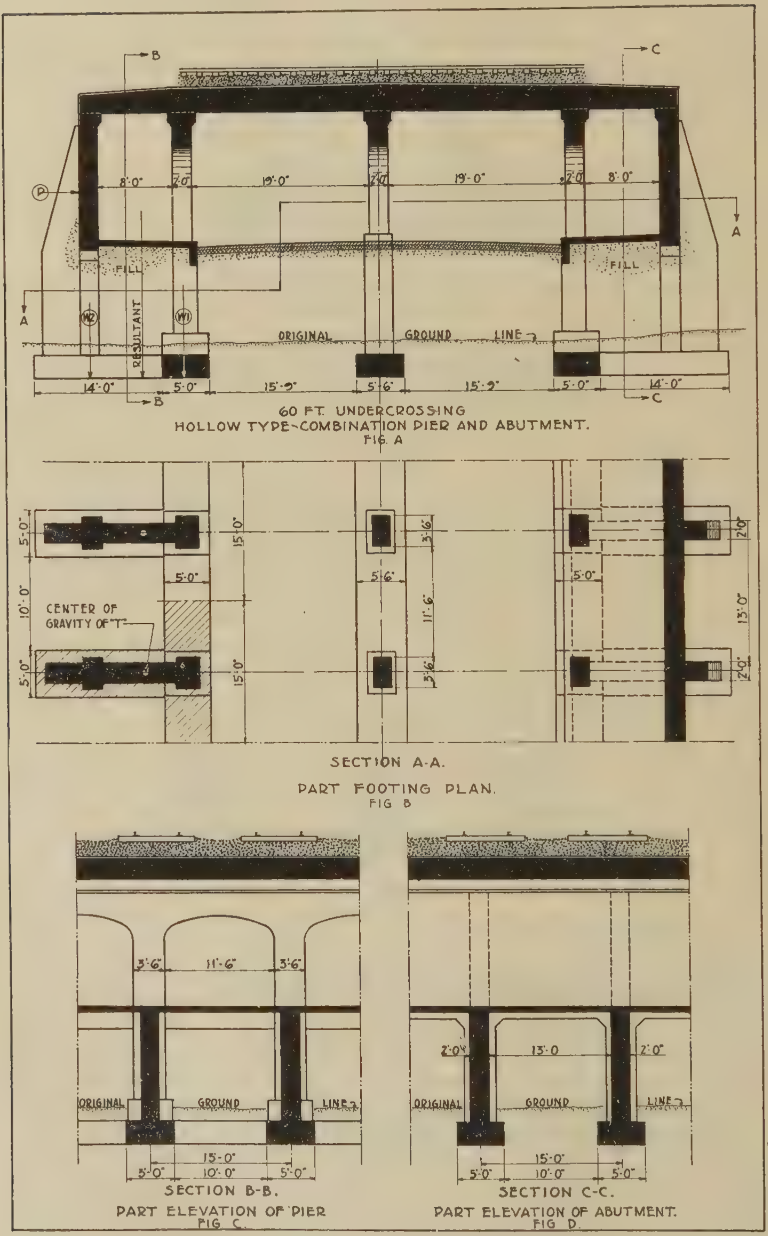


FIG. 4.—DESIGN FEATURES OF MULTIPLE-SPAN SLAB UNDER-CROSSING.

The total loads transferred to the center pier and each abutment were approximately equal, making the pressure on the base uniform. On the center section this amounted to 5.6 tons per sq. ft., and the slab for the 20-ft. invert span was designed to resist this pressure. In the same manner the remaining sections of invert were proportioned for their respective decreasing loads without exceeding the working stresses of concrete and steel. It could be said of the structure with its proportional inertia that it would adjust itself without failure in the event of an appreciable settlement not at all unlikely under the circumstances. Similar adaptations in concrete might easily be a controlling factor in the realization of projects of unusual magnitude.

Fig. 4 illustrates features in design of a multiple-span slab undercrossing where the controlling irregularity of the site was found to be an earth embankment of 10 to 12 ft. in depth supporting a highway over which four railroad tracks were to be carried. Since the profile of the track fixed minimum vertical clearances over the roadway and therefore minimum depth of floor, permission was granted to place column bents along curb and in the center of the roadway for reduction of span length and preclusion of girders between tracks.

Governed by the condition that the filled ground had inadequate bearing value, despite the fact that the load over full width of the street was to be divided into five reactions, it was imperative to carry foundations through fill to original ground. Instead of a five point distribution of reactions the design adopted reduced this to a three point concentration by combining abutment and curb pier bent to effect a series of T-shaped footings shown in plan by B, and in various sections of A, C and D.

The bent along the curb consists of a series of arched beams supported on columns 2 ft. x 3½ ft. spaced 15 ft. center to center. The footing for this bent is a continuous reinforced beam, 2 ft. in depth by 5 ft. in width, for the entire length of the bent. This footing would be required if the bent were a separate unit used for similar structure where roadway is founded on natural surface ground minus the fill. Instead of a 30-ft. high gravity abutment carried through fill to retain the railroad embankment and support the end slab reaction, a counterfort abutment was conceived. Each counterfort, 2 ft. in width, was placed on a line with corresponding columns of the curb bent and fashioned integrally with them by counterfort rib extension under the sidewalk shown in section at C. The longitudinal counterfort wall or facia abutment wall was carried only to the underside of the sidewalk leaving a wide open space between the counterforts for flow of railroad embankment, the major portion of its pressure being absorbed by the filled ground of roadway with equal diminution of the overturning moment of its pressure as represented by force *P* as shown on left of A, applied against longitudinal counterfort wall above sidewalk. The footing of this counterfort, also 5 ft. in width, was combined likewise with continuous footing of the curb pier bent, the first forming the stem and the latter the head of a series of T-shaped footing

increments, and so proportioned for maximum condition of loading that the resultant pressure of vertical reactions W_1 and W_2 of bent and counterfort respectively, combined with horizontal force P, is applied at the center of gravity of the tee resulting in uniform pressures throughout and fixed at a low unit to satisfy low permissible bearing value of original ground.

Railroad loading imposes limitations in concrete bridge design where the vertical clearance or headroom available is limited, where the foundations are earth and conditional requirements call for single long span. This condition may prevail if the structure is to be placed in the saddle of a street depression with steep descending grades in the approaches which would determine the exclusion of columns in the center of the roadway. The same restriction might result from other traffic regulations.

RAILROAD STRUCTURES.

A of Fig. 5 gives some indication of this limitation showing minimum sections of a reinforced-concrete arch spanning a 54-ft. street. The vertical clearance over the crown of the roadway is 14 ft.; over the gutter it is 13 ft. The depth of the crown section of the arch ring plus depth of full ballasted trach to top of rail is 4 ft. 1 in. The intrados is a five-centered curve approximating an ellipse. The arch curve was formed to the curve of loading or the pressure line to give the least amount of eccentricity of pressure from exceptional heavy locomotive loading placed over one-half of the span. Very heavy abutment sections are required and these sections would increase rapidly with increase of span. Weight alone is required to counterbalance the thrust of the arch and therefore the gravity abutment section is best adapted. The compressive stresses of the abutment are very low permitting lean mixture of concrete.

The multiple-span slab bridge is used most effectively where railroad tracks are to be elevated in the elimination of grade crossings. A first and foremost consideration in these projects is the determination of the floor depth of the bridges, since no other common factor affects the cost of the improvement in a greater degree. A reduction of the floor depth means an appreciable reduction in the amount of earth fill and in the amount of concrete in retaining walls, two items of great volume. A shallow floor carries with it a better solution of industrial siding connections and better adjustment and reduction in amount and length of street grades, if a combination of track elevation and semi-depression of streets is formed to be advantageous or necessary. Furthermore, a reduction of street grades lowers the amount of property damages, a considerable item in grade crossing eliminations through populous sections.

B and C of Fig. 5 represent different types of multiple-span concrete slab bridges. The slab of the former is reinforced in the one direction parallel with the longitudinal axis of the bridges while the slab of the latter is reinforced in four directions by bands of bars laid along all the center and diagonal lines between columns. The four-way arrangement is commonly known as the flat slab system.

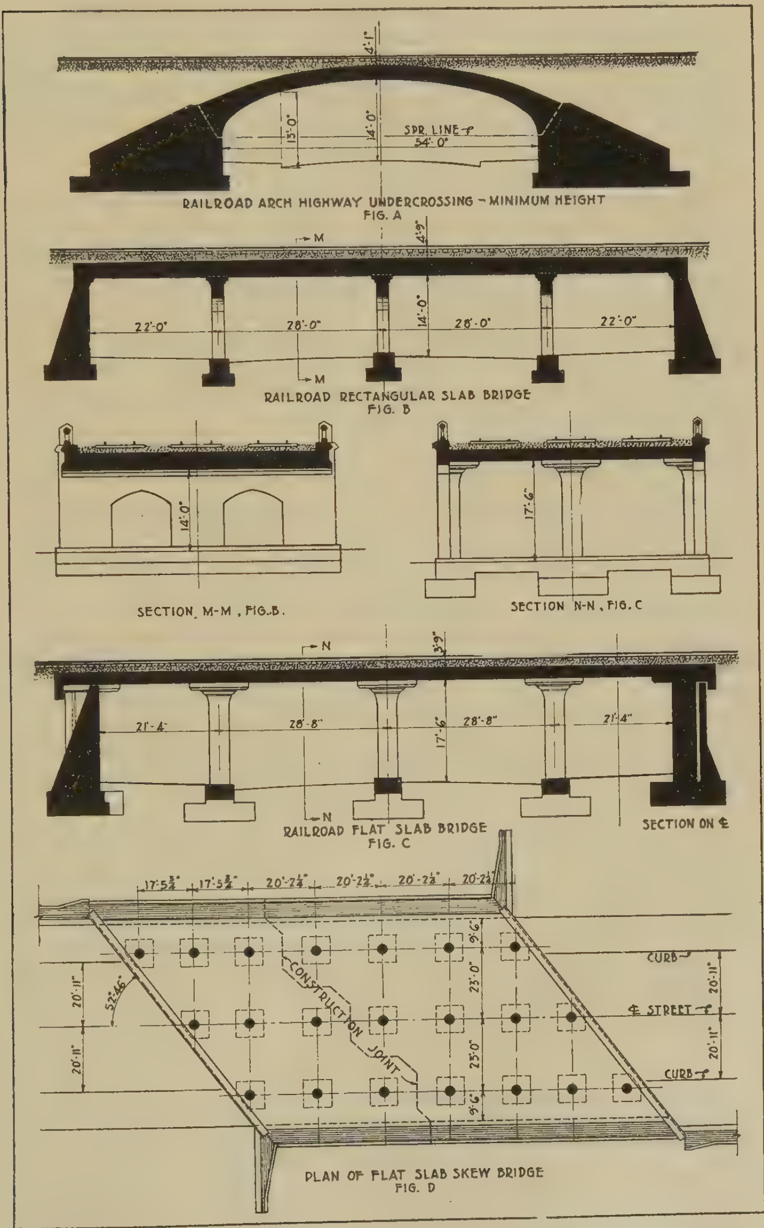


FIG. 5.—SECTIONS OF SINGLE AND MULTIPLE-SPAN CONCRETE BRIDGES.

A comparison of the two types indicates a distinct advantage off hand in favor of the flat slab type by a reduction of 1 ft. in the depth of the floor. Other comparisons have been made including a floor system composed of structural steel shapes encased in concrete that required the same depth of floor and a saving of one-quarter to one-half of the cost of the structure accrues in favor of the flat slab type. The amount and nature of the encasement controlled the variation in saving.

Other advantages of the flat slab bridge compared with all other forms of multiple-span slab bridges are summarily taken from a paper by the writer entitled "Reinforced Concrete Flat Slab Bridges" as printed in the 1918 *Proceedings* of American Concrete Institute. One advantage is embodied in the simplicity of both the formwork and arrangement of reinforcing steel. The steel placed in wide open spacings over practically unbroken flat surfaces insure a more positive placement than is found in the general beam and slab design. Another applies to its extreme shallow floor depth making it possible to lay the track in full ballast, which is a very important consideration in track construction; there are no girders projecting above the deck to encroach upon lateral clearances of the motive power. No noticeable vibration develops with the simultaneous passing of heavy locomotives and the rumbling noises common to structural steel bridges are very much subdued, are almost passive. The columns have wide, clear-cut spaces because of the fine integral relation between the slab and the columns; better visibility for drivers of vehicles and pedestrians is produced to bring about safe traffic regulations. Still another advantage exists, one that deals with a maximum fulfillment of adjusting the bridge to lineal irregularities and close vertical clearances; the slab can be placed parallel with the grade of the track and tilted to the grades of the street if necessary in transverse direction by placing the tracks at different levels which position may be found advantageous in case of industrial or yard tracks raised in elevation with main line tracks. An outstanding advantage of the flat slab is its uniform cross section and continuity of reinforcement. No type of multiple-span bridge construction, either steel or concrete is better proportioned to resist thermal changes because of this constant tensile stress everywhere and the paper mentioned above describes structures of great length which were built without expansion joints; the thermal changes take place at the construction joints of the slab in a multitude of minute movements that do not impair the strength of the structure.

CONDITION IRREGULARITIES OF RAILROAD STRUCTURE.

Conditional irregularities of the site become the most important consideration in the selection of small-type railway structures to be built on an established alignment; the design must be correlated with construction requirements in the maintenance of traffic assuming no interruptions thereto. This consideration is handled in a number of ways. Where the topography will permit, the alignment is shifted temporarily in order that

the bridge might be built clear of traffic, in part or in its entirety. Where the right-of-way is of limited width and the tracks cannot be shifted, a timber pile bent trestle of 12-ft. spans is driven under traffic and between these bents, after the excavation has been made, the abutment and piers only of the new bridge can be built. Long temporary through girders are often used to span out to out of the new abutment lines in order that the entire bridge may be built underneath. If no old girders are available and the only solution is the timber trestle there arises the exclusion of the flat-slab construction, for the reason that the floor system of the new bridge must be erected beyond the bridge site, either in units or in the whole, followed by a quick removal of the trestle stringers and the installation of the completed floor system on the new masonry during hours of least traffic.

A seven-track flat-slab bridge for grade crossing elimination is shown in D of Fig. 5, in which this consideration has been taken into account. The fact that the bridge angle of crossing is 52 deg. 46 min. did not complicate the design which indicates further flexibility of the flat slab. The work is divided into two parts along the construction joint as shown. This joint is placed without decrease of strength of the slab and so that the right half can be built first without interference with railway operations with the exception that this traffic must be confined temporarily to three tracks instead of seven in the vicinity of the bridge site. Those tracks which were temporarily abandoned are to be raised in elevation a maximum amount of eight feet; the remaining height necessary for undercrossing is obtained by a depression of the street. With the completion of the first half of the bridge the traffic is directed over the elevated tracks which permits the completion of the work. During construction, street traffic must be detoured until the first half of the structure is completed. The depression of the street is to be made simultaneous with the construction of the first half of the bridge.

A 12-ft. subway was built through an embankment under heavy railway traffic, as shown in A, Fig. 6. The embankment had been made on an alluvial deposit which necessitated pile foundation in support of the structure. Tho shift the tracks around the construction would require additional embankment and considerable trestle construction. Therefore, the temporary 12-ft. span trestle scheme built under existing alignment was found to be most efficacious; a further vital reason for this adoption was founded on the condition of great intensity of traffic, it being expedient to safer operating measure to keep the temporary construction on straight track.

With reinforced cantilever abutment and invert sections the structure was contracted to the least amount of space as between two bents of the 12-ft. temporary trestle; 12 ft. is the maximum span with 16-in. timber stringers. The piles for the trestle and invert foundation were driven between existing ties during hours of least traffic and the deck of the trestle was built during the same hours. Then followed the excavation of

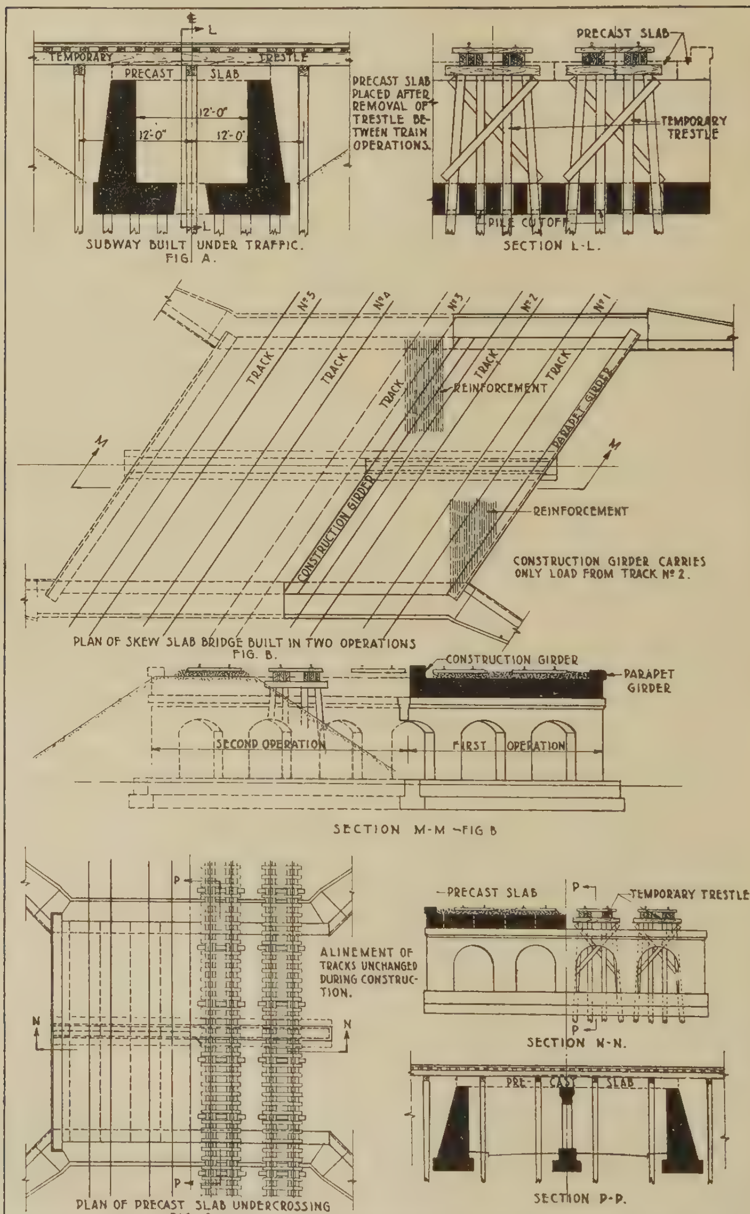

TEMPORARY
TRESTLE
PRECAST SLAB
12'-0"
12'-0"
12'-0"
PRECAST SLAB PLACED AFTER REMOVAL OF TRESTLE BETWEEN TRAIN OPERATIONS.
SUBWAY BUILT UNDER TRAFFIC.
FIG A.
PRECAST SLAB
TEMPORARY TRESTLE
PILE CUTOFF
SECTION L-L.
TRACK
REINFORCEMENT
PARAPET GIRDER
CONSTRUCTION GIRDER
CONSTRUCTION GIRDER CARRIES ONLY LOAD FROM TRACK № 2.
PLAN OF SKEW SLAB BRIDGE BUILT IN TWO OPERATIONS
FIG. B.
CONSTRUCTION GIRDER
PARAPET GIRDER
SECOND OPERATION
FIRST OPERATION
SECTION M-M - FIG B
ALINEMENT OF TRACKS UNCHANGED DURING CONSTRUCTION.
PLAN OF PRECAST SLAB UNDERCROSSING
FIG. C.
PRECAST SLAB
TEMPORARY TRESTLE
SECTION N-N.
PRE-CAST SLAB
SECTION P-P.

FIG. 6.

the embankment and the cutting of foundation piles. In the meantime the slab units were cast on platforms alongside the structure. After the completion of the abutments and backfill the trestle stringers were quickly removed and the precast slabs set in place by locomotive crane between train movements. Later the piles of center bent were cut off and their openings through the invert filled with concrete.

The same general scheme of operation was followed in the construction of a double 20-ft. span precast slab undercrossing. In this work gravity abutments could be used since the spacing of pile bents gives a full 12-ft. opening for the construction of abutment and pier sections. Here two of the tracks were put out of service during the construction of the first half to save cost of additional trestle.

A deviation from the general procedure of the foregoing cases is found in the construction of a skew type two-span slab undercrossing built in railroad embankment to carry five tracks. This bridge was built entirely in place operating Tracks No. 4 and 5 on the left of the plan while the right section of the bridge for Tracks No. 1 and 2 was being built. A trestle with short pile bents was driven under Track No. 4 to give natural slope to excavation made for the construction of the first section.

To give minimum thickness of slab for a skew bridge it was designed for the normal span and therefore the reinforcement was placed as shown in that direction; the live load skew span moment of wheel loading spread over a 13-ft. width of slab was equated to a uniform loading which formed the basis for computing the slab section. By the inclusion of a comparatively small amount of reinforcement in the parapet, it is formed to act as a girder to take the triangular loading of the elements of the slab, the reinforcement of which terminate, on the one end, in the parapet.

The construction joint of the slab was made along center line between Tracks 2 and 3 anticipating the use of a so-called construction girder, similar to the parapet girder, to carry the skew portion of the load from Track No. 2 during the construction of the second part of the bridge. The size of the construction girder was restricted by small space between ties. With this arrangement it was possible to keep the construction joints of the abutment and pier sections at the toe of the slope of excavation obviating the necessity of full trestle construction, or of showing of the track and therefore establishing safer train and construction operations. The reinforcement of the slab was placed through the construction girder to complete the slab, with the building of the second portion, in full strength exclusive of the construction girder capacity.

Overhead Highway Arch Bridges.

The reinforced-concrete overhead highway bridge in railroad construction of new lines or for grade crossing eliminations offers many interesting problems with reference to type selection and conditional requirements especially the one phase in grade crossing eliminations dealing with the

peremptory order of maintaining traffic with the least amount of interruption and danger of accident.

The importance of minimum depth of floor and headroom was emphasized in the case of highway undercrossings. In similar manner the headroom is often the controlling factor in type selection for overhead structures. A condition obtained in which the overhead highway arch could be designed for slightly shorter grades in the approaches than could be developed for the structural steel through-girder bridge. This is illustrated by A of Fig. 7. The depth of floor for the through-girder bridge was 3 ft. 6 in., a minimum obtained by placing the main supporting girders along the curb line projecting above the roadway with cantilever sidewalks on the outside. The minimum clearance for the steel girder bridge exists along its entire length but for the arch it exists only at the edge of railway clearance diagram over platforms. The shorter approach grades of the arch construction were secured by a long vertical curve at their intersection arch construction were secured by a long vertical at their intersection over the crown of the arch. For the steel structure, two vertical curves are necessary, one at either end of the bridge floor and these were made half the length of the vertical curve of roadway over the arch to correspond with same difference in amount of intersecting grade angles.

The slight advantage in favor of the arch was not the controlling factor in its selection. It was necessary to devise a scheme in construction that would insure continuous uninterrupted operations of suburban trains on all three tracks.

This was solved in different manner for two projects as shown by B and accompanying sections. In one case the centering was embedded in the arch ring and in the other it was a suspended type. In either case the arch ring was built in umbrella fashion on timber falsework over the platforms. The embedded centering consisted of a series of structural steel ribs fabricated to the shape of the arch ring. These ribs were embedded in the umbrella section and after the concrete had obtained sufficient strength the lagging was suspended by short bolts from the lower chord of the centering. Later in similar grade crossing elimination the lagging was suspended from two pairs of old girders which had been removed from long time service as railroad bridges.

The segmental and parabolic overhead types outlined in C and D respectively of Fig. 7, were built on new alignment: the former in a rock cut the latter on earth foundations. The barrel arch of the preceding case spanning four tracks without platform space is best suited for a site where minimum headroom is required. Where the established grade of the roadway gives considerably more headroom than required for standard clearances, the spandrel floor type of C and D becomes the choice. With excessive headroom, the barrel arch is subjected to unnecessary heavy load of fill, if the underside of the arch is fixed by standard clearances; if raised to avoid the heavy loading of the fill, the abutment and wing wall sections become excessively massive.

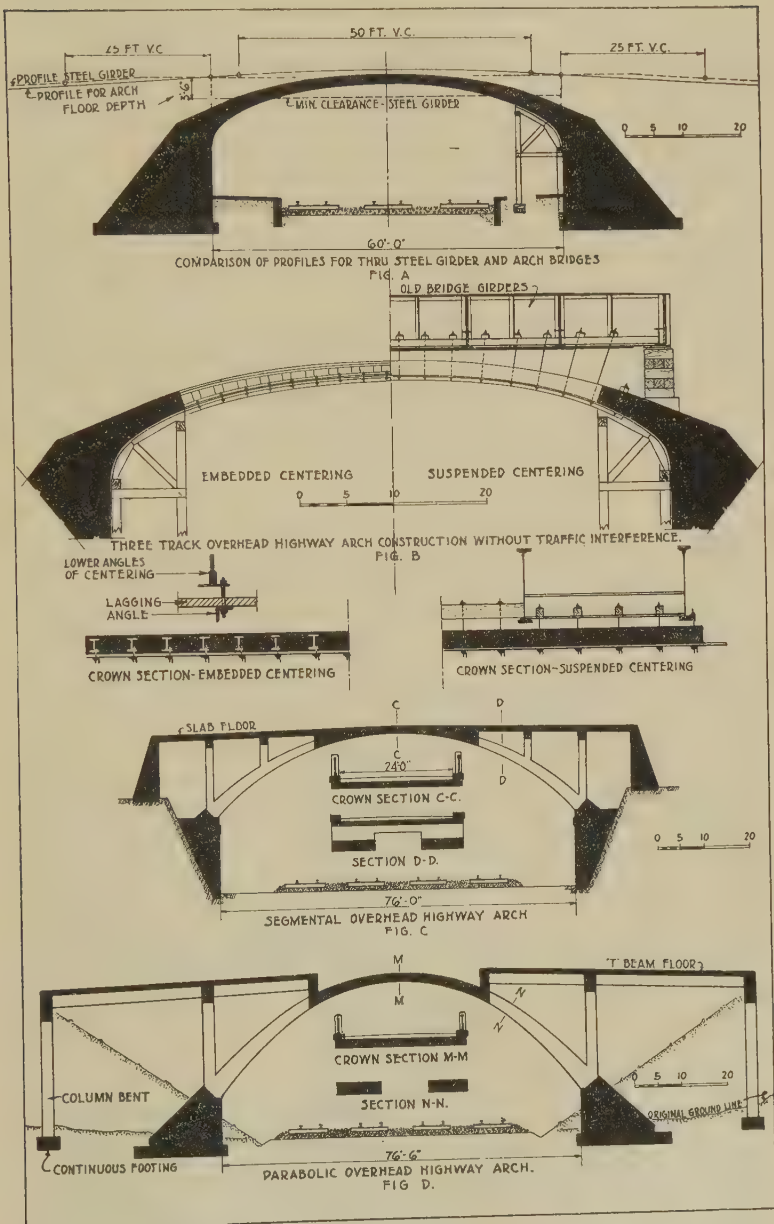

50 FT. V.C.
25 FT. V.C
25 FT. V.C.
PROFILE STEEL GIRDER
PROFILE FOR ARCH
FLOOR DEPTH
MIN. CLEARANCE - STEEL GIRDER
0 5 10 20
60'-0"
COMPARISON OF PROFILES FOR THRU STEEL GIRDER AND ARCH BRIDGES
FIG. A
OLD BRIDGE GIRDERS
EMBEDDED CENTERING
SUSPENDED CENTERING
0 5 10 20
THREE TRACK OVERHEAD HIGHWAY ARCH CONSTRUCTION WITHOUT TRAFFIC INTERFERENCE.
FIG. B
LOWER ANGLES
OF CENTERING
LAGGING
ANGLE
CROWN SECTION - EMBEDDED CENTERING
CROWN SECTION - SUSPENDED CENTERING
SLAB FLOOR
C
D
24'0"
CROWN SECTION C-C.
SECTION D-D.
0 5 10 20
76'-0"
SEGMENTAL OVERHEAD HIGHWAY ARCH
FIG. C
M
"T" BEAM FLOOR
N
CROWN SECTION M-M
SECTION N-N.
COLUMN BENT
0 5 10 20
ORIGINAL GROUND LINE
CONTINUOUS FOOTING
76'-6"
PARABOLIC OVERHEAD HIGHWAY ARCH.
FIG D.

FIG. 7.

The parabolic type is submitted to show a balancing of these factors. The arch is divided into two ribs over the haunch section; the crown section is solid and replaces floor construction. The parabolic shape of the arch ring was determined by this heavy concentration at the crown of the arch ribs, augmented by the single reactions of the tee-beam floor which comprise all the arch loading. The concentration of the center column bent of the floor system is brought forward to the face of the abutment in order to reduce the eccentricity of the arch thrust on the abutment and therewith reduce its size. The end span of the tee-beam floor served another purpose by replacing four massive retaining walls, 40 ft. in height; the end column bent is buried in surrounding fill of the approaches.

The wide pier of the multiple-arch construction may be prohibitive for overhead bridge crossing of several parallel railway lines or series of tracks in yard, station or terminal layout. The long span tee-beam roadway deck supported on slender columns, similar to the cross-section of A, Fig. 8, was used to fulfill the limitations of space between each series of tracks. The span of tee-beam was 50 ft.

OVERHEAD COLUMN SLAB BRIDGES.

Again the consideration of expansion joints is a pertinent subject. Unlike the flat-slab construction which has a constant tensile stress in every direction, the tee-beam type offers varying degrees of tensile resistance. There arises the difficulty of transferring the movement due to temperature changes from the larger through the smaller members, as from the deep beams through thin slabs. These forces should be carried directly through the main members of the floor system to the points of support. Regardless of the length of the structure, the thin slabs of the tee-beam should be substantially reinforced in direction parallel with the beam. For spans of 50 ft. it is a futile effort to attempt to take up temperature movements by sliding joints. Plates of steel, copper, zinc or lead and multiple layers of fabrics or combination of these different plates have been used to reduce frictional resistance. When the bearing covers a wide area there is the danger that the sliding medium will conform to possible uneven surface of bridge seat or that frictional resistance greater than the strength of the beam in shear will develop causing a cracking of the beam adjacent to the support and sliding joint. Rocker bearings have been used instead of sliding joints for long spans; this deviation has questionable efficacy and requires, in manner of the sliding joint, a support of considerable size and stability.

The type of construction under discussion has been built in total length of 179 ft. without failure, the stress resulting from the movement being taken by bending of the columns. For length of structures of 300 ft. the stress would be exceedingly large and the incorporation of an expansion joint is advisable. Because of the doubtful action and bulky detail of the sliding or rocker joint they were discarded for a double row of

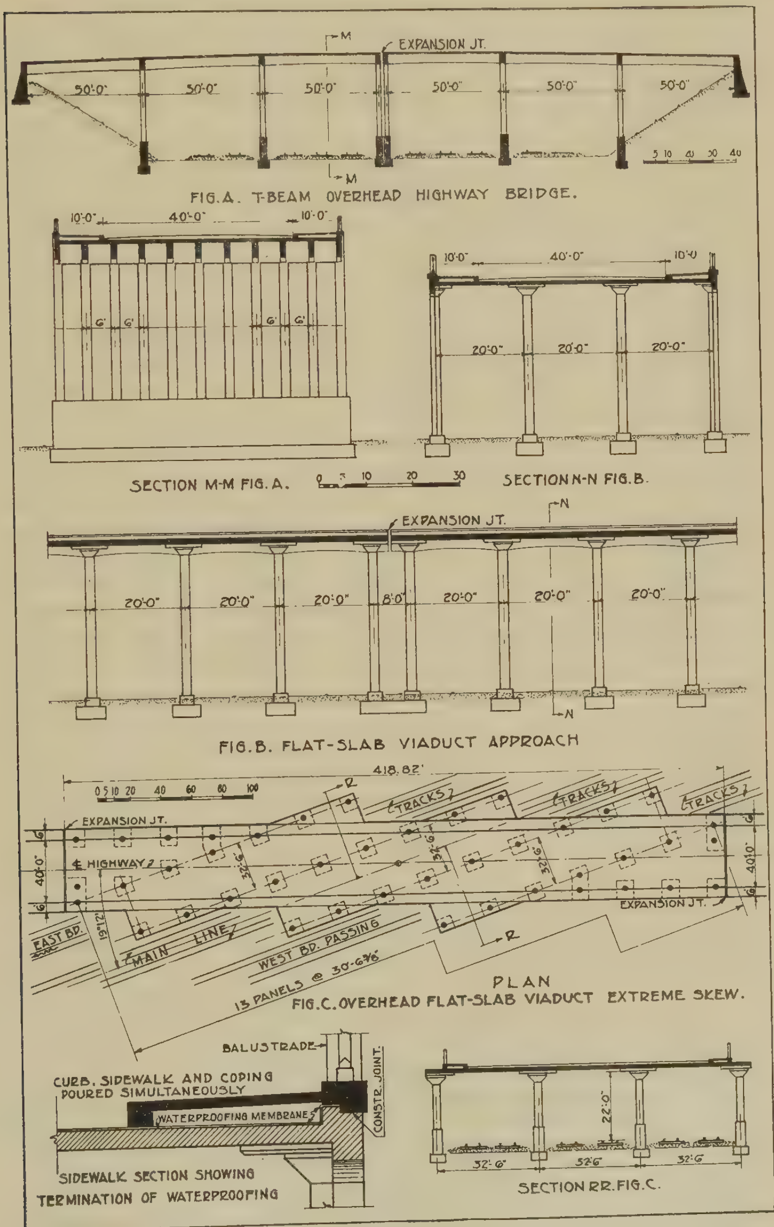

EXPANSION JT.
50'-0"
FIG. A. T-BEAM OVERHEAD HIGHWAY BRIDGE.
10'-0"
40'-0"
6'
SECTION M-M FIG. A.
20'-0"
SECTION N-N FIG. B.
EXPANSION JT.
8'-0"
FIG. B. FLAT-SLAB VIADUCT APPROACH
418.82'
TRACKS
EXPANSION JT.
℄ HIGHWAY
32'-6"
40'-0"
EAST BD.
MAIN LINE
WEST BD. PASSING
13 PANELS @ 30'-6 7/8"
PLAN
FIG. C. OVERHEAD FLAT-SLAB VIADUCT EXTREME SKEW.
BALUSTRADE
CURB, SIDEWALK AND COPING POURED SIMULTANEOUSLY
WATERPROOFING MEMBRANE
CONSTR. JOINT
SIDEWALK SECTION SHOWING TERMINATION OF WATERPROOFING
22'-0"
32'-6"
SECTION R-R. FIG. C.

FIG. 8.

columns dividing the structure into two separate and distinct units of 150 ft. in length with stability in flexure to resist the movements due to temperature. Each 50-ft. beam is supported directly on a column, thus eliminating a heavy beam cap over the columns and effecting simple and direct structural lines.

The advantages claimed for the flat slab in railway bridge construction are repeated in application to the lighter loads of highway bridges. This is exemplified in all phases in design of flat slab structure built over six tracks on the extreme acute angle of crossing of 19 deg. 21 min. as shown by C of Fig. 8.

The shallow floor depth was of vital importance in this case since it gave minimum length to the approaches built over perfectly flat ground. The skew span over six tracks is 418.82 ft. In what other type of multiple span construction could this length be covered without an expansion joint and how could an expansion joint be made to act effectively along center lines of skew? The slab over this length was cambered easily in form construction to shape of a vertical curve for intersecting approach girders.

The size of the slab panels of the section over the tracks was determined in the one direction by placing column lines between each of three pairs of tracks on 32-ft. centers. In the other direction lengthwise with the track the spacing of the columns is 30 ft. 6⅝ in. forming an almost square panel. This spacing was determined by placing on the curb line of the viaduct the diagonally opposite end columns of the two rows adjacent to and parallel with the future outside tracks. The total distance between these two end columns, measured along the line of track, one line projected on the other, is divided into 13 equal spaces of 30 ft. 6⅝ in. The slab depth is 15½ in.

In addition to the columns of the center section placed between the tracks, there is another row of four columns along the curb line which forms several irregularly shaped panels in order to carry out the triangular portion on each end of the center section to a row of columns normal to the center line of the roadway in line with the last column of the outer row parallel with the tracks. With the exception of these few irregular panels all other panels of the center section are 30 ft. 6⅝ in. x 32 ft. This arrangement of placing the panels of the center section normal to the track forms six triangular portions of the slab which project beyond the sidewalk or balustrade line. These projections however, considering the acute angle of crossing are comparatively small, and little of the bridge floor is unused.

The columns between the tracks are protected to a considerable extent in case of sideswipe in derailment by placing them on a three-foot wide pier six feet above the top of rail. The piers are reinforced by four lines of old rails. This high pier has another function since alluvial soil deposit required pile foundation; it acts as a spread footing in distribution of loads on wood pile foundation spread out from column to column in narrow confines between the tracks precluding the necessity of driving piles and constructing footing courses beneath the tracks.

A hypothetical case, B of Fig. 8, has been taken to show the possibilities of flat slab construction for viaduct approach to a bridge 60 ft. in width. The 20 x 20-ft. panels give a very economical spacing of columns. A facia girder adds rigidity to outside row of columns and also lends pleasing architectural effect. It might be found that this type of construction would be more economical than an arch fill and retaining wall approach for a height exceeding 18 ft. No paving base would be required with the flat-slab construction and a permanent pavement would be ready for traffic with the completion of the structure; for an earth fill this permanent pavement would not be realized until the year following the completion of the bridge with cessation of fill settlement. The space under the viaduct approach could be utilized for some purpose—perhaps, the parking of automobiles. In the lower lefthand corner of this plate, a sidewalk section is given to show the method of terminating the waterproofing membrane and the construction joints of this part.

Conditional Irregularities of Larger Type of Structures.

To this point examples of salient features in the design of the moderate size reinforced-concrete bridge have been given in measure of the potentialities of concrete as they may be developed in adjustment of footing plans to special geological irregularities; as they may be developed in adjustment and balance of the superstructure to lineal or topographical irregularities; and finally as these potentialities may be developed in correlating the design and construction methods of the small type railroad structures in grade crossing elimination to the exacting requirements of maintaining traffic during construction.

Examples of larger type bridges follow to give further measure of the potentialities in concrete and to point out the necessity of correlating features in design and such limitations in construction as are found in the possibilities of plant development, flood conditions and traffic maintenance requirements in congested districts—pedestrian, vehicular and railway combined.

Harrison Avenue Bridge.

The Harrison Avenue Bridge was built in Scranton, Pa., to connect two sections of the city separated by a deep ravine. A photograph of the completed structure, a sectional elevation and plan featuring plant layout and construction methods are combined in Fig. 9.

The main line of the Delaware, Lackawanna and Western R. R. is located near the top of the ravine on the one side and the Lehigh and Wyoming Valley R. R., a third-rail electric line is located in a similar position on the other side. The electric line is 12 ft. higher in elevation than the steam railway. It is an ideal location for arch construction from standpoint of suitable foundations since rock outcropped everywhere. The total length to be spanned by the three arches was determined by distance out to out of railway property lines. Various lengths for the center span were tried and a balance struck by the selection of a 200-ft. arch flanked by two 75-ft. arches over the railways.

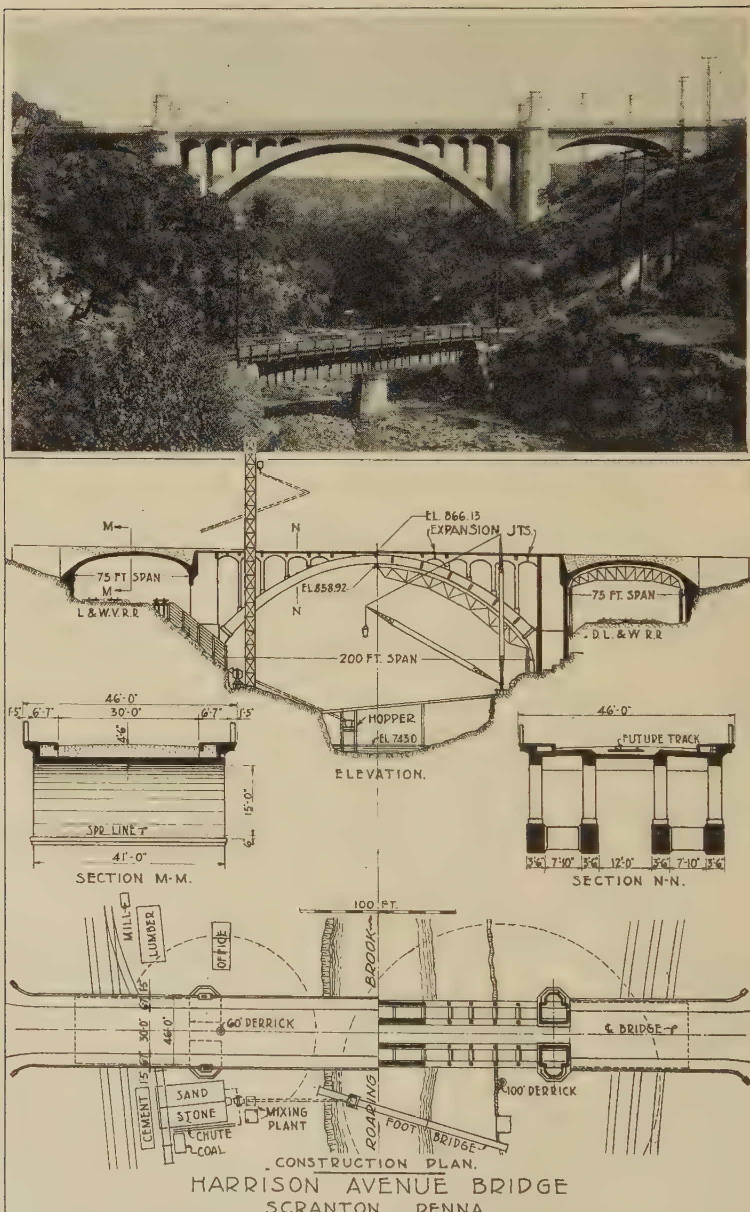

EL. 866.13
EXPANSION JTS.
M
N
75 FT SPAN
L.&W.V.R.R.
EL.858.92
200 FT. SPAN
75 FT. SPAN
D.L.&W.R.R.
HOPPER
EL 743.0
ELEVATION.
46'-0"
30'-0"
6'-7"
1'-5"
4'-6"
15'-0"
SPR. LINE
41'-0"
SECTION M-M.
FUTURE TRACK
3'6" 7'-10" 3'-6" 12'-0" 3'-6" 7'-10" 3'-6"
SECTION N-N.
100 FT.
MILL
LUMBER
OFFICE
BROOK
60' DERRICK
℄ BRIDGE
100' DERRICK
SAND
STONE
CEMENT
MIXING PLANT
CHUTE
COAL
ROARING
FOOT BRIDGE
CONSTRUCTION PLAN.
HARRISON AVENUE BRIDGE
SCRANTON PENNA.

FIG. 9.

In adjusting the arches to the lineal irregularities of the site, consideration was given to the location of a satisfactory ledge of rock for the main arch abutment; to a limitation of the height of the intermediate piers; to railroad clearances and to an approximate balance of the end arches to compensate for the difference in level of the railroads. The photograph is a view of the west elevation while the elevation drawn is of the other side. The right end of the structure, as considered by the drawing, was lowered as much as possible and the other raised to adjust this difference. To some extent this raising and lowering was controlled by approach grades. It resulted in the placing of one-half of the bridge floor on a 1 per cent grade and the other half on a 2 per cent, both ascending to the left. The 1 per cent grade on the high side and the 2 per cent grade on the low side gave camber to the long horizontal lines of the parapet and a better architectural effect than the straight average grade over the full length of the span. Symmetry in structural shape was the basic thought behind all considerations in lineal adjustment. The structure is symmetrical about its center lines. In construction the main arch span was divided into two units, elsewhere there were four symmetrical units permitting a substantial re-use of forms.

The thin, deep arch rib arranged in two pairs was the composition decided upon for the main arch span to give minimum weight in construction, a consideration of importance in the design of the centering for a 200-ft. span over deep ravine. A conservation of concrete materials is not always the most economical basis of design. In this case they were to be hauled a considerable distance and transferred to a second line for delivery. It was evident that the handling of a minimum amount of materials over and about the railroad would give the most satisfactory results. Therefore the conservation of concrete materials did influence, in a general way, the design of other features of the bridge.

The floor slab is carried on spandrel arch beam and column construction, 2 ft. 10 in. in width, identically repeated over the four main arch ribs forming three floor bays; the center bay was depressed to provide for future street car track. A transverse beam at each column line divides the center bay into a series of square panels reinforced in two directions to give the 10-in. slab of the smaller side bays reinforced in one direction. Transverse beams are placed in side bays adjacent to expansion joints to increase the lateral stability of the floor.

The barrel type arch was used for the 75-ft. spans over the railroad tracks. The outer abutments are small—merely a facing of the rock. The thrust on the other side is taken by the intermediate pier called "Pilaster Box" in larger detail of B, Fig. 10. It is so named because this unit is in composition with the ornamental pilaster. The boxes are reinforced and anchored at one end of the longitudinal walls to the main arch abutments to resist the overturning moment due to the thrust of the 75-ft. arch; this thrust is applied against the longitudinal walls of the boxes at a considerable distance above their bases. The end section

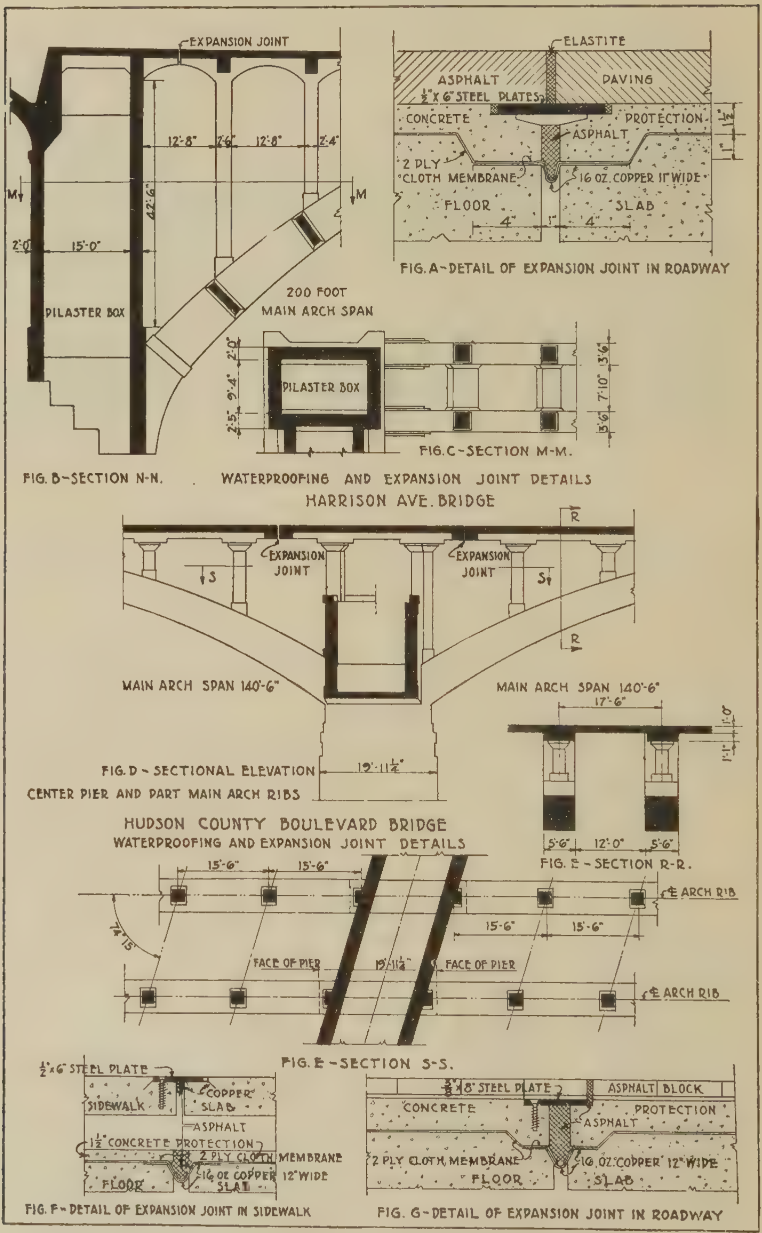

EXPANSION JOINT
12'-8"
2'-6"
12'-8"
2'-4"
42'-6"
15'-0"
2'-0
PILASTER BOX
200 FOOT
MAIN ARCH SPAN
ELASTITE
ASPHALT
PAVING
½"X 6" STEEL PLATES
CONCRETE
PROTECTION
ASPHALT
2 PLY
CLOTH MEMBRANE
16 OZ. COPPER 11" WIDE
FLOOR
SLAB
FIG. A-DETAIL OF EXPANSION JOINT IN ROADWAY
PILASTER BOX
FIG. C-SECTION M-M.
FIG. B-SECTION N-N.
WATERPROOFING AND EXPANSION JOINT DETAILS
HARRISON AVE. BRIDGE
EXPANSION JOINT
EXPANSION JOINT
MAIN ARCH SPAN 140'-6"
MAIN ARCH SPAN 140'-6"
17'-6"
19'-11¼"
FIG. D - SECTIONAL ELEVATION
CENTER PIER AND PART MAIN ARCH RIBS
HUDSON COUNTY BOULEVARD BRIDGE
WATERPROOFING AND EXPANSION JOINT DETAILS
5'-6"
12'-0"
5'-6"
FIG. E - SECTION R-R.
15'-6"
15'-6"
℄ ARCH RIB
FACE OF PIER
19'-11¼"
FACE OF PIER
℄ ARCH RIB
FIG. E - SECTION S-S.
½"x6" STEEL PLATE
COPPER
SIDEWALK
SLAB
ASPHALT
1½" CONCRETE PROTECTION
2 PLY CLOTH MEMBRANE
16 OZ COPPER 12" WIDE
FLOOR
SLAB
FIG. F - DETAIL OF EXPANSION JOINT IN SIDEWALK
⅜"x8" STEEL PLATE
ASPHALT BLOCK
CONCRETE
PROTECTION
ASPHALT
2 PLY CLOTH MEMBRANE
16 OZ. COPPER 12" WIDE
FLOOR
SLAB
FIG. G - DETAIL OF EXPANSION JOINT IN ROADWAY

Fig. 10.

of the arch is reinforced in transverse direction to carry the entire thrust to the four walls. The pilaster boxes receive no counter stability from the floor system of the main arch on account of the expansion joint adjacent to the pilaster box which forms a clear-cut opening through the floor.

The construction program was centered about the use of structural steel centering for all three arch spans. The steel centers for the main span were built in the form of a three-hinged arch and were composed of two ribs 3 ft. 6 in., center to center, to be placed under each concrete arch rib. Two concrete ribs were formed simultaneously requiring four steel ribs. Heavy timber bents carried steel I-beams on which the shoes of the centers were placed. Wedges were placed underneath and behind the shoes to provide adjustment of centers to correct elevation. After two ribs had been poured and set, the entire system of centering and lagging was lowered several inches as a unit and rolled over for the construction of the remaining two ribs. For the 75-ft. spans, six trusses of the Pratt type were built, three being required for the construction of one-half width of each arch. The top chord of these trusses followed the curve of the arch; the bottom chord could not be made flat, on account of clearance requirements of the railroads. The centers were erected by the 60- and 100-ft. derricks shown in construction plan which were also used with chuting system to deposit concrete.

A gravity concrete plant was used. A delivery track was built on a trestle, parallel to the main track of the L. & W. V. R. R. Stone and sand bins, having a capacity of 300 and 200 tons, respectively, were built on the side slope under the trestle, so that hopper cars, carrying concrete material were dumped directly into the bins. A measuring hopper, discharging into the mixer was built at the foot of the bins. The cement house was built at the track level alongside the top of the bins and cement was handled to a platform at the measuring hopper through a chute. With this arrangement, all concrete material was handled to the mixer by gravity. The plant had a mixing capacity of 100 cu. yd. of concrete per day with the services of six men. A tower and chutes were used for concreting the arches and floor system. The concrete for the structure on the opposite side of the ravine was passed through the tower to a hopper built on the foot bridge and then handled to place by the 100-ft. derrick with buckets. No operation exceeded the 100-yd. daily capacity of the plant. The main arch rings were divided into thirteen blocks or voussoirs, corresponding to the distance between concrete struts which were poured in six operations.

As heretofore described, the through arches form separate structural units in equilibrium. Any one unit could be carried to completion independently in comparatively small operations. It was possible to subdivide the work to be carried on by constant working force and the contractor devised his plant layout to reduce this to a minimum number. Mention was made of the expansion joints of the main arch floor adjacent

to the pilaster. It will be noted that two other joints were provided near the quarter points of the span. The conclusions prompting this arrangement were drawn from the following experience:

EXPANSION JOINTS, WATERPROOFING AND DRAINAGE.

In 1911, the Delaware, Lackawanna & Western R. R. completed two large concrete structures known as the Delaware River and Paulins Kill viaducts. In connection with the design of these structures, after an extended investigation had been made of existing structures of similar type, it was decided to construct the viaducts without expansion joints in the floor system. No precise data could be obtained at that time concerning the extent of the seemingly small rise and fall of the existing arches. It was thought that most of the cracks, attributed to temperature changes, were due to shrinkage of the concrete in the first few months of construction and that if the viaducts with foundations carried to rock were built in such a manner as to minimize this action, the expansion joints could be eliminated.

The plan and sequence of construction were outlined in the following manner: the main arch rings were built in voussoir blocks about ten feet in length measured along the axis of the arch ring and spaced two feet apart. The blocks were allowed to set seven days before the two foot openings or keys were poured and twenty-eight days after the last key was poured the arch center was removed. This eliminated an undue settlement of the arch ring due to shrinkage and the theoretical crown elevation for which the arch was designed was further insured by cambering the arch center for its deflection under the ring load. No part of the floor system was built until the arch centers had been removed. The floor was built in alternate sections, but the reinforcing steel in the slab was continuous for the full length of the viaduct, which, in conjunction with the elasticity of the superstructure, was deemed adequate to resist the rise and fall of the arch ring without the formation of large cracks. It was expected that a number of very fine cracks would result from this movement, but these were not considered so objectionable as the leaky expansion joints that were in evidence on most of the structures inspected.

An examination of the Delaware River and Paulins Kill viaducts, after they had been in service for some time, disclosed the fact that large transverse cracks had developed at the crown of each of the two spandrel arches adjacent to every pier of the viaduct. This indicated that there had been an appreciable drop of the main arch rings due to fall in temperature. (See Fig. 11.) Over the crown, the spandrel arches were in compression and no cracks developed while those adjacent to the pier were in tension. It may be here noted that the floor system was built during the summer months and therefore the movement in the main arch ring due to rise in temperature, would bring the floor system back to its normal position, or, if there were a movement beyond this it would be so slight as to make its effect on the floor system directly over the crown negligible,

as compared with the effect of a fall in temperature on the floor adjacent to the piers. The appreciable vertical movement of the heavy arch ring for a rise and fall of temperature is transferred to the comparatively light floor system which must also resist its own movement in a horizontal direction. These movements cannot be resisted by the addition of any amount of reinforcing steel and expansion joints must therefore be provided.

In view of the above observations and conclusion, it was decided in designing the Tunkhannock and Martins Creek viaducts, that expansion joints would be provided in the floor system at the piers and quarter points of every span. The quarter points of the span are approximately the

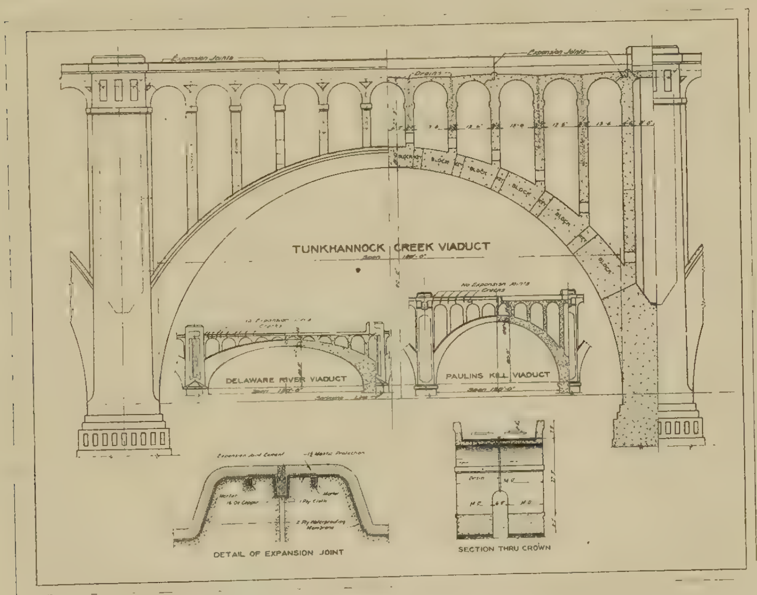


FIG. 11.

points of contraflexure in the arch ring. For a fall in temperature the extrados of the arch ring between the points of contraflexure is in compression, beyond these points in tension and vice versa for a rise in temperature. The expansion joints divide the superstructure into three parts, separated at the points of contraflexure, each section acting in compression or tension as the case may be. This arrangement has proved successful for both viaducts, which have been in service for the past ten years.

The sliding surfaces of the expansion joints of the big viaducts were lined by an asphaltic membrane about 3/16-in. in thickness. Although no visible failures have occurred in the floor slab adjacent to these expansion joints due to excessive friction, they have been experienced in flatter and lighter arches which prompted a further development. As the

ratio of the rise to the span of an arch increases, the movement due to temperature changes increases likewise.

The cantilever joint has replaced the sliding joint in the most satisfactory manner. Each section of the floor system is free to move up or down, forward or back, regardless of the movement or rigidity of the adjacent sections. In the floor panel to receive the joint, two cantilever sections are substituted for the single span. The expansion joint is formed by a clear opening separating the ends of the cantilever sections. The expansion joint adjacent to the pilaster of the Harrison Avenue bridge and joints adjacent to the center pier of the Hudson County Boulevard bridge, to be explained later, are shown in detail of Fig. 10.

Originally expansion joints leaked badly, gradually, as the full significance of their function was realized, methods for overcoming this deficiency were developed. It was found that the system of waterproofing and drainage had to be correlated with the provision made for expansion. First, an elastic waterproofing membrane to cover the entire floor area was necessary and proper provision for carrying the membrane over the joints and terminating it in places of least water accumulations had to be devised. Then a further precaution in leading the water away from the joint and off the floor at close intervals was deemed expedient so that no large amount of water is carried over long distances or down a single spout. Details shown in Fig. 11 illustrate the manner in which these requirements were carried out in planning the floor of the Tunkhannock Creek viaduct. Attention is called to the provision for terminating the waterproofing in reglets along parapet walls at base of rails; also to the 1½-in. mastic protection laid over the membrane to prevent abrasion caused by movement of ballast.

The space given to track ballast over railway bridge floors helps materially in the development of these protective provisions for expansion joints. For highway bridges they are confined to very little space under the pavement as shown in Fig. 10 by the two examples detailed. Steel plates spanning expansion joints exposed to highway traffic are soon loosened by impact of heavy vehicles. In the details referred to above these plates are placed under the pavement to avoid this failure. Some maintenance of the paving joint above the plates is anticipated, but this will be of little consequence compared with the repairs necessary to exposed plates.

CHEMUNG RIVER BRIDGE, CORNING, N. Y.

No condition affecting structural measures gives greater apprehension to the builder of bridges than flood conditions. These menacing interruptions were anticipated in the construction program of a multiple-span reinforced-concrete arch bridge built over the Chemung River for the City of Corning, N. Y.

The structure is 752 ft. long and is composed of seven three-centered segmental barrel skew arches 92 ft. 3-in. spans and 11 ft. 3-in. rise. Dimen-

sions of width are shown in section of Fig. 12 which also has a view of the completed structure and a diagram of construction plan.

Three streams, draining about 4,200 sq. mi., come together near Corning to form the Chemung River. These streams are flashy, and floods come quickly and subside almost as quickly. In the maximum recorded flood of March 14, 1918, the river rose 19 ft. The rate of rise has at times been 1 ft. an hour. In computing waterway the length of the bridge 752 ft. was set as the distance between the dikes, which have contained past floods, and the roadway was set at an elevation which gave an aggregate waterway between piers below springing lines sufficient for the record flood, leaving the arch spaces above springing lines as a factor of safety in case of greater floods.

With the foundation conditions, 2 to 6 ft. of gravel and clay, then stiff clay down to 16 ft., and then hardpan, the first problem in design was to choose between shallow footings on piles and deep foundations protected by cofferdams. Because of the flood conditions and the favorable bottom, the advantage was in favor of piles. The compactness of the ground lead to the choice of cast-in-place concrete piles. This type of foundation had several outstanding advantages over other general methods: (1) Compared with wooden piles, if the wooden pile could be driven in the compact soil, only one-half the number were required thereby reducing the size of the footings; (2) wood piles would also have necessitated a deeper foundation at some of the piers; (3) compared with a precast pile a time element was considered (no time was required for curing of the cast-in-place pile and the work of driving could be started immediately after the excavation); and (4) reinforcing rods could be inserted immediately after the piles were poured, to bond the footing course to the piles. Additional stubs placed in the footing course lapped the vertical bars in the pier shaft and also in the face walls of the abutments. This arrangement was effective in resisting eccentric thrusts in constructing the arch ring, in addition to the bond formed or integral combination made between superstructure and substructure.

In planning the equipment, the frequency of the floods suggested mobility and retreat and the high ground on the north bank of the river, with the protection afforded by the dike, offered the haven of safety. Therefore the construction of the bridge was planned around a locomotive crane, with a 60-ft. boom, and a construction plant as indicated by Fig. 12. A derrick, having a 75-ft. mast and 70-ft. boom was placed within reach of the stockpiles, the cement and steel store houses and the concrete mixing plant.

Work started by excavating for the south abutment. Building and grading a ramp from the top of the bank the crane worked its way down to the river's edge, and thence across on tracks laid in the bed of river during low water, driving the sheeting and making the excavations for the river piers as it proceeded. Immediately after the excavation was completed the pile-driving equipment was lowered into the hole and the

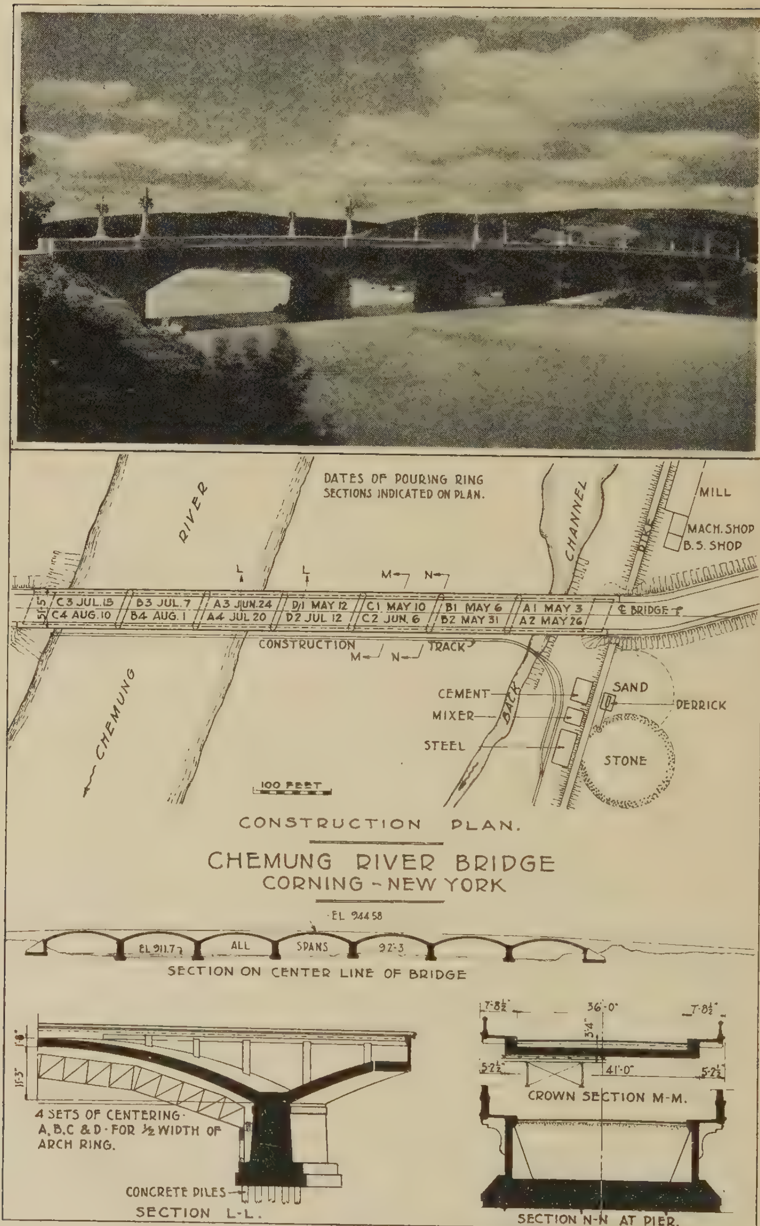

DATES OF POURING RING SECTIONS INDICATED ON PLAN.
RIVER
CHEMUNG
CHANNEL
BACK
MILL
MACH. SHOP
B.S. SHOP
C3 JUL.15
C4 AUG.10
B3 JUL.7
B4 AUG.1
A3 JUN.24
A4 JUL 20
D1 MAY 12
D2 JUL 12
C1 MAY 10
C2 JUN.6
B1 MAY 6
B2 MAY 31
A1 MAY 3
A2 MAY 26
℄ BRIDGE
CONSTRUCTION TRACK
CEMENT
MIXER
STEEL
SAND
DERRICK
STONE
100 FEET
CONSTRUCTION PLAN.
CHEMUNG RIVER BRIDGE
CORNING - NEW YORK
EL 944.58
EL 911.7
ALL SPANS 92'-3
SECTION ON CENTER LINE OF BRIDGE
4 SETS OF CENTERING A,B,C & D FOR ½ WIDTH OF ARCH RING.
CONCRETE PILES
SECTION L-L.
7'-8½"
36'-0"
41'-0"
5'-2½"
CROWN SECTION M-M.
SECTION N-N AT PIER.

FIG. 12.

driving of the piles commenced. It was estimated that 15 piles could be driven in a working day and this progress was made in driving 633 piles in 42 working days. About two days were required to move the driver from pier to pier. The driving schedule was interrupted for a short period in the early part of November due to an unprecedented flood for this particular month.

Work on the piers followed the pile driving. The foundations for the river piers were poured before floods of any consequence occurred, but the pier shafts were completed between floods, with a very narrow margin of time in each operation. Work on the piers was advanced so that the superstructure could be started from the north end and be completed on the high ground during spring months to the river's edge before the June flood. Fortunately, due to the unusual drought this flood did not materialize but the work was developed accordingly to the predetermined plan which contemplated that high water would cause delay but would not cause serious damage to the work or plant.

For the construction of the superstructure, the work was divided into two parts along the center line of the bridge. Structural steel centering was used for the arch construction. One unit, forming a half-width of arch barrel, was composed of two ribs made up of four separate Pratt truss sections. These trusses carried the lagging as shown in crown Section M-M of Fig. 12.

The full width of pier had the necessary stability to resist the overturning moment of the horizontal thrust of one unit of the arch centering fully loaded without any counterbalancing thrust on the opposite side of the pier. This was one condition of construction operation. It also had the required stability to resist the other maximum construction stress, a combination of the thrust of the centering on one side of the pier with the thrust of a completed half on the other side of the pier.

The sequence of arch construction is shown in Fig. 12. The rings were poured in order named using four sets of centering marked A, B, C and D for one-half width of the arch ring. Each one was a continuous operation pouring alternately from either end. Including a 9-in. lift over the half area of the pier, there were 160 cu. yd. in each operation, poured in eight hours with a 1-yd. mixer. The short time between arch ring operations was employed without reduction of force in the construction of the parapet walls, sidewalk and balustrade. When the last half of the arch ring was poured Aug. 10, the parapet walls were almost completed on all the spans and much of the sidewalk and railing was completed. In this manner operations were distributed over the entire work so that a maximum force could be economically employed up to the last three weeks. Remarkably fast work was done with the centering erection and concrete placing in the arches. Within one month after the arrival of the centers four halves of arches had been poured. The centering in course of erection for this first operation is shown by C of Fig. 13. The construction track in normal river condition is shown by insert, B, and

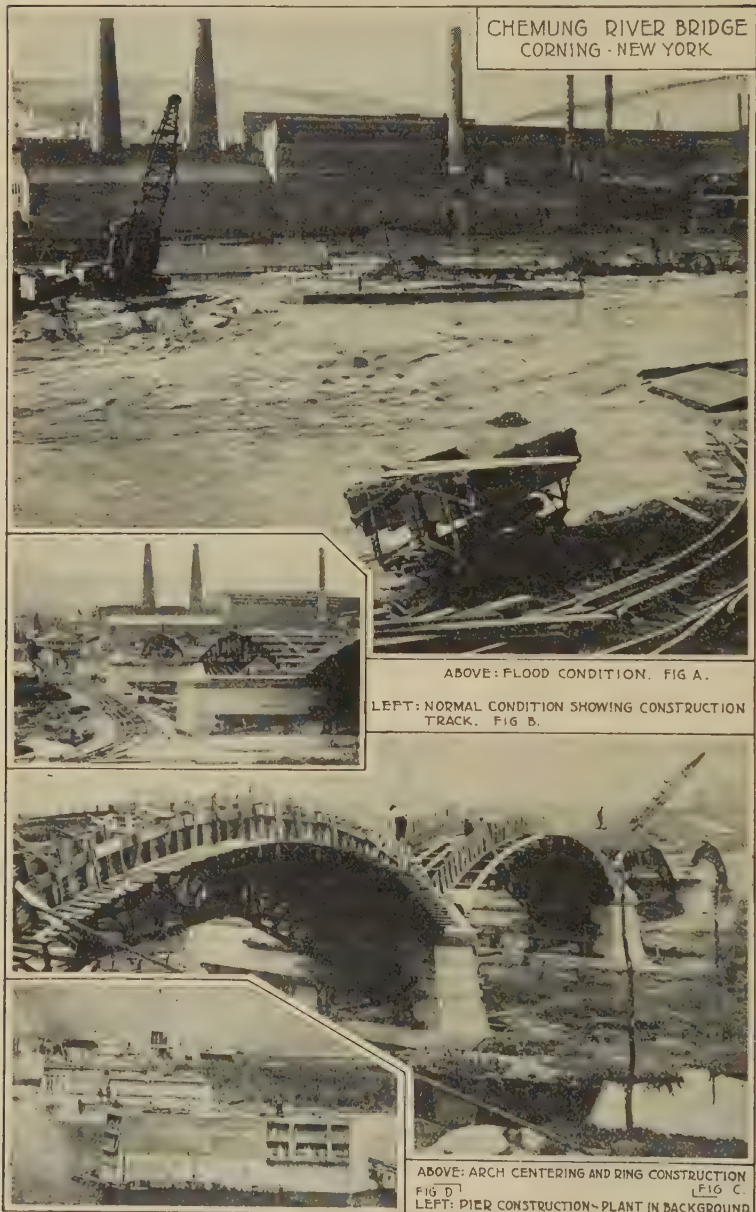

CHEMUNG RIVER BRIDGE
CORNING - NEW YORK
ABOVE: FLOOD CONDITION. FIG A.
LEFT: NORMAL CONDITION SHOWING CONSTRUCTION TRACK. FIG B.
ABOVE: ARCH CENTERING AND RING CONSTRUCTION
FIG C.
LEFT: PIER CONSTRUCTION - PLANT IN BACKGROUND.
FIG D

FIG. 13.

a flood condition taken from the same point of view is shown by photograph, A. This particular flood condition preceded the arch construction and came about without warning and with such rapidity during the night that the contractor's plan of retreat of equipment was foiled.

Riverside Drive Bridge, Binghamton, N. Y.

In the construction of the five-span multiple skew arch bridge built over the Chenango River at its point of confluence with the Susquehanna River, there were the same flashy flood conditions as existed during the construction of the Chemung River bridge. Both rivers are tributaries of the Susquehanna. The arches too, are similar, being the five-centered full-barrel type with 86-ft. span and 13-ft. rise. A photograph of the completed structure and a diagram of construction plan are shown in Fig. 14.

The method of construction as outlined is not the method contemplated by the plan and specifications, which were prepared in the office of the city engineer. All sections of the structure were well designed. The first requirement of the specifications, affecting control of the construction program, was formulated by a conclusion that the arch ring should be poured in its full width, 58 ft. On account of the volume of concrete of each ring a provision was made to pour the arch in three sections, a crown and two haunch sections. The construction joint separating these sections was composed of nine offset planes arranged parallel with the skew axis of the arch. Each plane was perpendicular to the pressure lines of the arch assuming an action parallel with the longitudinal axis or center line of the bridge. A second requirement of the specifications called for the full completion of the centering for the entire structure before any concrete in any of the arches was to be poured. A third provision was that the centering of an arch should not be struck until at least thirty days after the completion of the arch.

In the final design of the centering the question arose as to whether or not these articles of control did not create unnecessary hazards. They evidently presumed that the centering would be built largely of timber. Assuming that they did create unnecessary hazards the alternative steel centering plan as outlined in Fig. 14 was developed for comparison with a timber centering. Taking the affirmative side of the question, it was contended that extraordinary, perhaps insuperable difficulties would be experienced in holding or anchoring timber bends in the river bottom with only a few feet of gravel overlying bedrock under three of the arches. With the usual force of carpenters employed on a bridge of this magnitude, considerable time would be required to construct 27,000 sq. ft. of centering, and much expense and trouble would be encountered in maintaining the lagging during the construction on account of long time exposure to the elements exclusive of flood conditions. If a flood condition occurred during the arch construction and any part of the centering under load of the arch ring was washed out there was the danger of

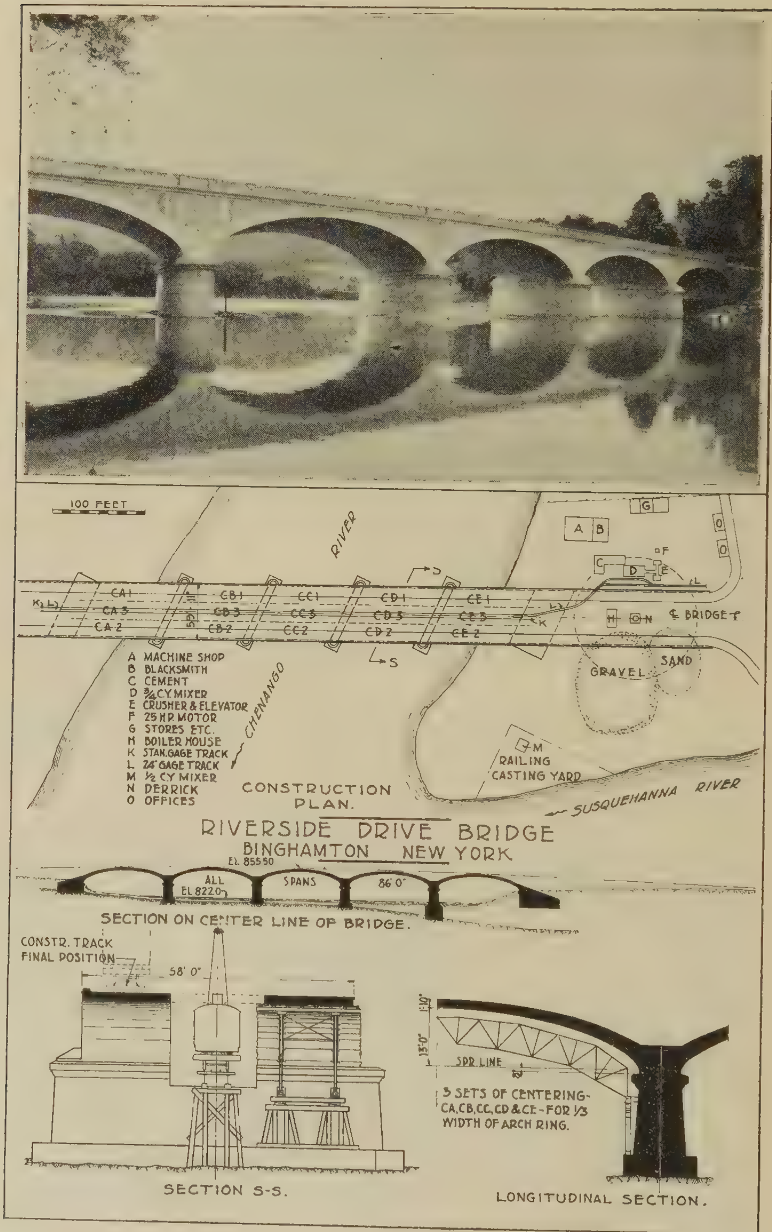

100 FEET
RIVER
CHENANGO
CA1
CA3
CA2
CB1
CB3
CB2
CC1
CC3
CC2
CD1
CD3
CD2
CE1
CE3
CE2
S
K
L
H
N
℄ BRIDGE
A B
G
O
C D E
F
GRAVEL
SAND
M
RAILING
CASTING YARD
SUSQUEHANNA RIVER
A MACHINE SHOP
B BLACKSMITH
C CEMENT
D ¾ C.Y. MIXER
E CRUSHER & ELEVATOR
F 25 H.P. MOTOR
G STORES ETC.
H BOILER HOUSE
K STAN. GAGE TRACK
L 24" GAGE TRACK
M ½ C.Y. MIXER
N DERRICK
O OFFICES
CONSTRUCTION PLAN.
RIVERSIDE DRIVE BRIDGE
BINGHAMTON NEW YORK
EL. 855.50
ALL SPANS 86' 0"
EL. 822.0
SECTION ON CENTER LINE OF BRIDGE.
CONSTR. TRACK FINAL POSITION
58' 0"
SECTION S-S.
1'-10"
13'-0"
SPR. LINE
3 SETS OF CENTERING-CA, CB, CC, CD & CE-FOR ⅓ WIDTH OF ARCH RING.
LONGITUDINAL SECTION.

FIG. 14.

failure because the piers did not have the stability to resist the overturning moment of the thrust of a completed arch on one side of the pier only.

The alternative plan provided a long-span construction trestle designed to carry a standard locomotive crane and a narrow-gage gasoline locomotive. All construction operations were to be carried on from this trestle. Concrete was to be transported in buckets from the mixing plant to all parts of the work on flat-top cars hauled by the gasoline locomotive and deposited in the forms by the crane. The crane would handle one full rib of the centering fabricated in the yard. The plan called for a division of the arch rings into three ribs parallel with the longitudinal axis of the arch. Each rib was to be poured in continuous operation and the contention was made that this would result in a more substantial arch than one poured in three full sections with transverse joints in the thin arch ring. Five sets of centering covering one-third of the width of the arch were to be used in the first position along the right side of the structure, in the second position along the left side, and, finally, in the middle section after the crane track had been placed on the completed section of the arch. An analysis of the pier proved it to have the required stability under any condition of construction loading. The bent supporting the centering on pier footing was to be bolted to the pier to preclude danger of a wash out. With the completion of one-third of the arch all piers would be stabilized. After a careful consideration of the two schemes, the alternative plan with structural steel centering was adopted. The contractor arranged his schedule of ring construction to complete five sections in one month so that the first section of centering used could be moved bodily to its second position after the completion of the fifth and so on in continuous operations. In the meantime work on the parapet walls and balustrade was carried on.

B of Fig. 15 illustrates the easy manner of erecting one full rib of the centering; the insert, A, shows centering in place ready for the lagging; C is a view of the working trestle during pier construction and D shows stability of the structure during construction and a thwarting of the flood hazard.

Hudson County Boulevard Bridge Plaza

The Hudson County Boulevard is the one north and south bound thoroughfare extending the length of the county along the Palisades on the New Jersey side of the Hudson River opposite New York City. It is one of the most important thoroughfares of the Metropolitan District, used as the Lincoln Highway carrying all the traffic south and west from New York City. Its importance will be greatly increased when the Holland vehicular tunnels now under construction under the Hudson River between Jersey City and lower New York City are opened.

The alignment of the boulevard is fairly straight until it reaches the point, in the vicinity of Journal Square, Jersey City, where it is

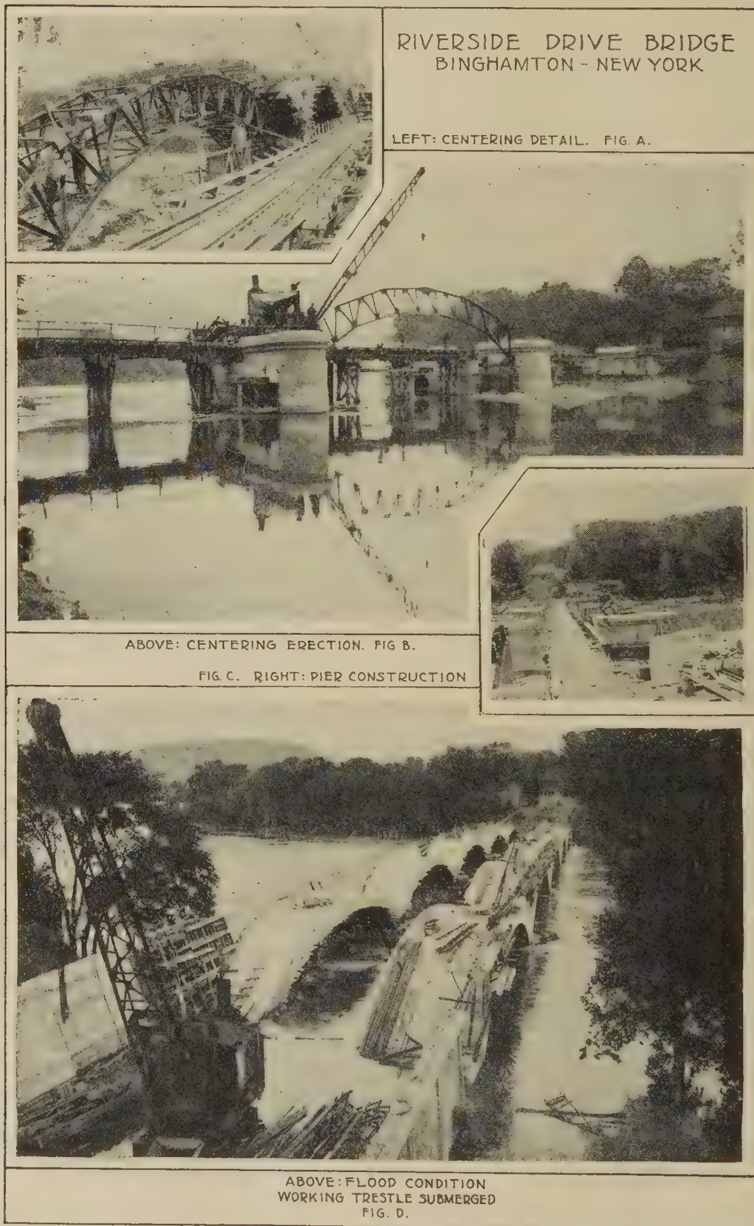

RIVERSIDE DRIVE BRIDGE
BINGHAMTON - NEW YORK
LEFT: CENTERING DETAIL. FIG. A.
ABOVE: CENTERING ERECTION. FIG B.
FIG. C. RIGHT: PIER CONSTRUCTION
ABOVE: FLOOD CONDITION
WORKING TRESTLE SUBMERGED
FIG. D.

FIG. 15.

carried over a depression, in which the tracks of the Pennsylvania and Hudson and Manhattan railroads are located, on a three-span structural steel deck bridge, 342 ft. long and 60 ft. wide, carrying a 40-ft. roadway and two 10-ft. sidewalks. The height of the roadway over the tracks is 50 ft. The improved roadway beyond the bridge is 60 ft. in width. This bridge was built in 1894 when the boulevard was no more than a bridle path. Due to the heavy traffic, an excessive vibration developed throughout the entire structure, making necessary an immediate reconstruction.

The boulevard has a very tortuous alignment in either approach to the bridge. In a northerly direction the roadway curves to the left, starting close to the abutment, through 31 deg. Likewise, going south it begins to curve to the right immediately as it leaves the south abutment through 77 deg., then follows one full block of tangent and another turn left through 46 deg. In this elbow is the center of great traffic activity, which is due in large measure to the excellent rapid transit service to New York, afforded by the Hudson and Manhattan "tube trains," from the Journal Square station which is located just east of the south end of the bridge.

In 1912, the first year of operation, the tube trains carried 4,000,000 passengers from the Journal Square station. In 1922 this was increased to 24,000,000 and since then there has been a further increase. These figures give a fair indication of the growth and magnitude of traffic in this section. The great bulk of this passenger traffic is handled to and from the station by buses, private cars and taxicabs along the boulevard. A small portion is carried by street car service to the terminal on the station concourse. Before the general use of buses, this was the main carrier to the station.

All the bus lines with one exception terminated near the south abutment of the old bridge discharging and loading passengers on the bridge and around the curve immediately adjacent thereto. This was for the convenience of passengers in reaching the 15-ft. concourse which was the only avenue of access from the boulevard to the station. The buses from the north discharged passengers on the bridge and turned around in two narrow streets forming Journal Square. The Bergen Avenue line used these streets also and turned at the street intersection north of the bridge which is also the turn around for buses from the south on the boulevard. The two principal lines, north and south on the boulevard carry monthly over 2,000,000 passengers. To complicate the situation, there is the through Lincoln Highway and local travel on the boulevard. The maximum eight-hour count of vehicles over this bridge four years ago was 15,000. These factors, the tortuous alignment of the boulevard, the narrowness of the bridge roadway which gave great intensity of vehicular travel, the inaccessibility of the station and the enormous growth of travel have caused a traffic congestion comparable in importance and magnitude to the larger traffic problems of the Metropolitan District. The design of a new bridge at this point had to contemplate first a solution of this traffic congestion.

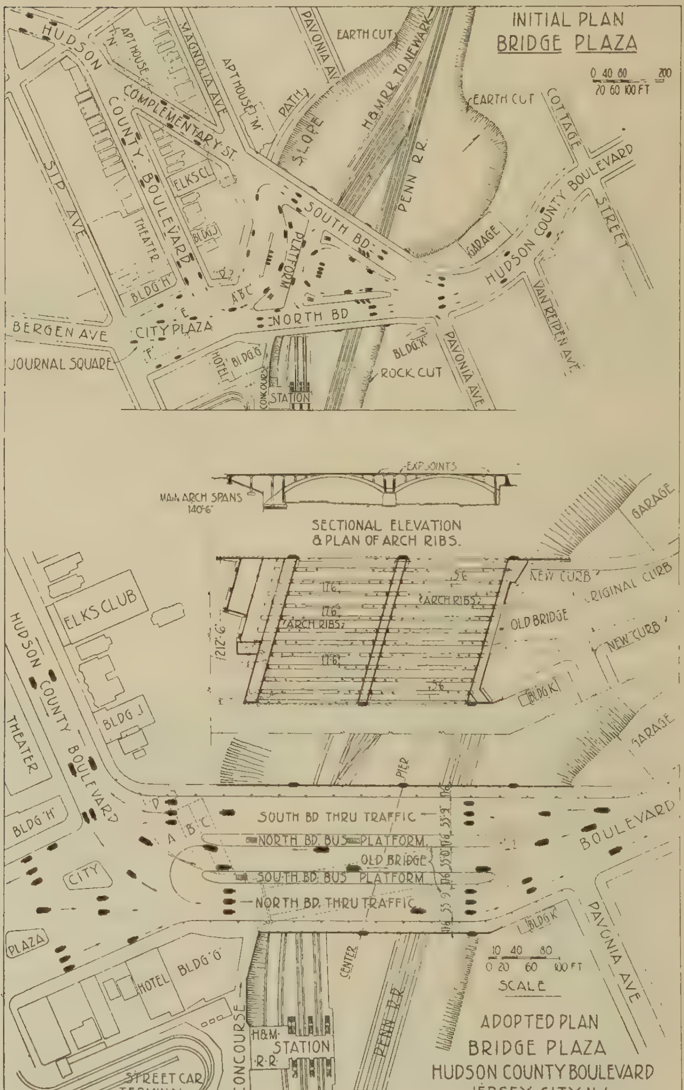

INITIAL PLAN
BRIDGE PLAZA
HUDSON
COMPLEMENTARY ST.
MAGNOLIA AVE
APT HOUSE
PAVONIA AV
EARTH CUT
H.&M.R.R. TO NEWARK
PENN R.R.
COTTAGE STREET
HUDSON COUNTY BOULEVARD
COUNTY BOULEVARD
SIP AVE
THEATER
ELKS CL
SOUTH BD
PLATFORM
NORTH BD
BERGEN AVE
CITY PLAZA
JOURNAL SQUARE
HOTEL
BLDG G
BLDG H
BLDG K
ROCK CUT
PAVONIA AVE
VAN REIPEN AVE
GARAGE
STATION
CONCOURSE
SECTIONAL ELEVATION
& PLAN OF ARCH RIBS.
EXP JOINTS
MAIN ARCH SPANS 140'-6"
ARCH RIBS
NEW CURB
ORIGINAL CURB
OLD BRIDGE
ELKS CLUB
BLDG J
HUDSON COUNTY BOULEVARD
PIER
SOUTH BD THRU TRAFFIC
NORTH BD BUS PLATFORM
OLD BRIDGE
SOUTH BD BUS PLATFORM
NORTH BD THRU TRAFFIC
BOULEVARD
CITY
PLAZA
HOTEL
BLDG G
CENTER
PENN R.R.
H&M R.R. STATION
STREET CAR TERMINAL
SCALE
ADOPTED PLAN
BRIDGE PLAZA
HUDSON COUNTY BOULEVARD
JERSEY CITY, N.J.

FIG. 16.

A plaza greater in magnitude than the usual bridge replacement was deemed necessary.

Two solutions were developed in the course of study as shown in plan (see Fig. 16), which are differentiated by titles "Initial Plan" and "Adopted Plan." The scale of the adopted plan is twice that of the initial plan. They are drawn in aparallel position, however, to facilitate the relating of buildings, streets, railroads, the old bridge and other topographical features to both plans. The old bridge is shown in position by dotted lines in the adopted plan by which the circuitous alignment of the boulevard may be traced.

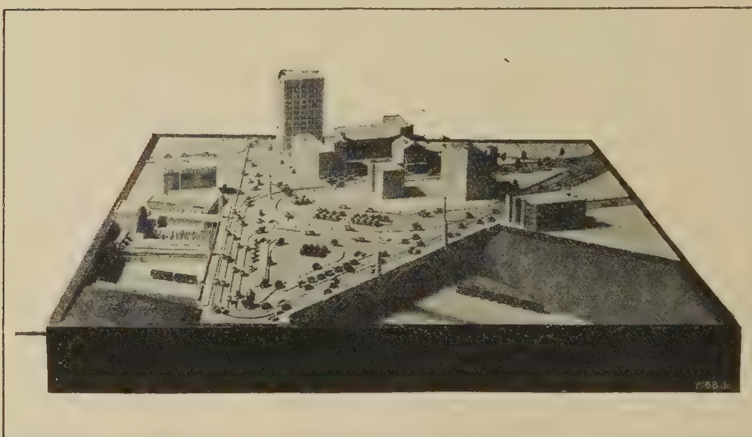
The Initial Plan.—This plan contemplated a correction of the alignment of the boulevard and a solution of traffic difficulties of this section by projecting a new street forming the hypotenuse of a triangle with the existing alignment to divert part of the traffic from the principal turn of the boulevard at the south abutment where the traffic jam occurred. The new 90-ft. street cleared the six-story Elks' Club House and two large apartment houses marked "M" and "N". The buildings that would be razed in the line of this new street between the three buildings mentioned are two-story frame dwellings which are incongruous with the rapidly-increasing property values. The new or so-called complementary street was carried across the cut of the railroad and intersected the present roadway of the boulevard just beyond the northerly abutment of the old bridge. The two roadways over the tracks, the one in position of the old bridge, the other, the complementary street, formed the sides of an A-shaped bridge plaza 200 ft. in width at the northerly abutment or apex, 525 ft. in width at the base along the top of the railroad cut, the length or altitude being 465 ft.

Traffic Movements.—This A-shape made possible a one-way movement in traffic control. Within this space three large platforms were placed to separate the traffic southbound on the westerly leg and northbound on the easterly leg or roadway of the bridge. The two platforms parallel with the roadways form a center plaza wherein the north bound vehicles or buses could be drawn away from the through traffic. Passengers would be unloaded on the one platform and loaded from the other, the vehicles turning within the area. The large platform forming the crossbar of the "A" would be used as a receiving and unloading platform for southbound passengers. The buses would operate along the lower or curved side of the platform. The upper or straight edge of the platform would be used for private vehicles from the north turning with the general traffic movement. The sides of this big platform would be used for unloading passengers from vehicles continuing north and south beyond the plaza. There is a parking space in the center between platforms and along the side of the platforms for taxicabs and momentary stops of private vehicles. Private vehicles could also park along the roadway sides of the platforms.

Stairways were planned to lead from the three island platforms to two longitudinal and one main or transverse passageway built directly

underneath and suspended from the bridge floor. The transverse passageway was in line with the existing passageway to the station and was to be built underneath it to the width of 20 ft. Other stairways from the various corners of the plaza connect with these passageways to make it possible to reach the station or the three island platforms from any point on the surface without crossing the roadways. Hudson County and Jersey City authorities jointly would remove the buildings in Journal Square, thus giving Bergen Ave. proper access to the boulevard.

Plaza Model.—To better visualize the entire project a mechanical model (see Fig. 17) was made to a scale of 1 in. equal to 20 ft. The



HUDSON COUNTY BOULEVARD BRIDGE PLAZA.
BIRDS-EYE VIEW OF MECHANICAL MODEL OF INITIAL PLAN.

FIG. 17.

roadway of the model was made of laminated wood 3/16-in. in thickness. Slots were cut in the roadway in the center along the principal lines of traffic movement making loops of each separate route. Under the ends of the roadway were slipped sprocket wheels or cone pulleys so set that the face of each slip came directly under the turn in the corresponding slot in the street. The toy pressed pewter automobiles were secured by hair pins to chains acting as cableways and operating around the sprocket wheels. By turning the propelling crank to left of model box all lines moved around the plaza along their proper routes showing continuous movement without interference.

Adopted Plan.—Balanced against this scheme was the alternative plan which was finally adopted, contemplating a straight bridge, parallel sides, 212½ ft. in width. A reduction in scope of the undertaking may be given as a principal reason for the decision made in favor of the

straight bridge. Legal complications did arise with question of authority of county to project a new street through the city.

The deck of the bridge is composed of three roadways separated by two safety isles or platforms, each 17½ ft. in width. In addition there are two main sidewalks of the same width. The westerly roadway is to be used for southbound through traffic, the easterly roadway for northbound through traffic and the center roadway with safety isles are to be used as the bus terminal. The direction of travel along the three roadways is indicated in plan by arrow marks. Traffic regulations will be necessary at either end of safety isles as the southbound bus leaves on turn and northbound bus enters the center roadway, and vice versa. This regulation may be corrected later on by operating buses in through service for the full length of the boulevard. The arrangement draws the buses out of traffic and will correct the traffic jam, but not as effectively as the initial plan because the southerly turn from the plaza leads to the existing 60-ft. wide boulevard which must carry the traffic in either direction as heretofore. It is true that in the initial plan this condition prevails also, but at a considerable distance from the center of action giving more space for a spreading out or reduction of traffic intensity.

Work in construction of the new bridge started in October, 1924, and will be completed about the first of September of this year.

The bridge is composed of two main arches of 140½-ft. span and 23-ft. rise. These spans consist of 13 separate arch ribs 5½ ft. in width and 12 ft. apart. One arch spans all the tracks of the Hudson and Manhattan, a third-rail electric system, and the other spans the Pennsylvania R. R., not electrified. Superimposed on these arches is a series of concrete columns which support a reinforced-concrete roadway slab. Beyond the tracks the roadway slab is carried by columns variable in height from the footing courses, the maximum of which is approximately 50 ft. The general arrangement of the arch ribs is shown in sectional elevation and plan of Fig. 16.

The angle of crossing of the bridge with the railroad tracks is 74 deg. 15 min. The complications inherent with the design and construction of a skew arch bridge were precluded by an arrangement in which each arch rib is placed in a normal position. The small offset of each arch pier and abutment shaft formed along pier and abutment wall lines has been developed into a pleasing architectural composition.

The arch rib type forms another function in the scheme of traffic regulation. It will be possible to suspend from and place between the arch ribs as many passageways as will be required for passengers going to and from the station to the buses. Stairways from the bus platforms will lead to these passageways with further connection to train platforms and station. At present two longitudinal and three transverse passageways are contemplated. Provisions for one transverse passageway have been made in the upper hollow portion of the center pier and the other two over the abutment sections. With these passageways pedestrians and

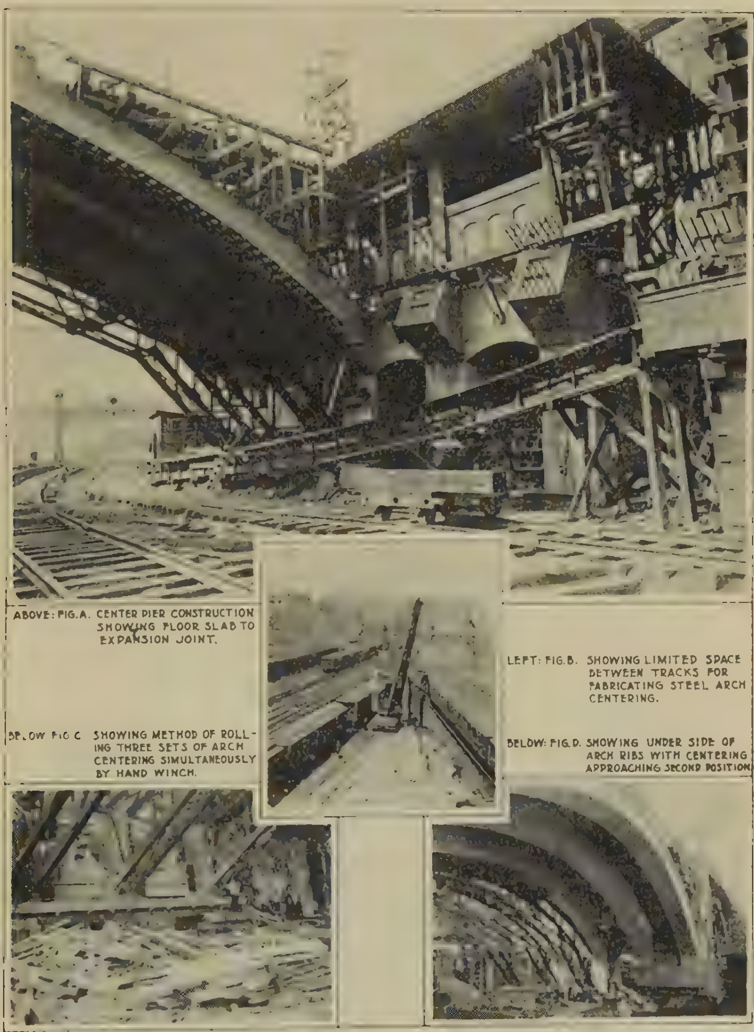
passengers may reach any corner of the plaza, the safety isles to buses and the station without crossing the roadways jammed with vehicular traffic. Any surface regulation of combined vehicular and pedestrian traffic across a width of 176 feet of roadway would considerably retard both and the danger of accident to pedestrians would not be eliminated as it would be with the passageways.

The pier shafts have the same width of main arch ribs and have not been enclosed below the springing line. A connecting arch forms a passageway on the track level so that in the future it will be possible to construct a platform around the pier for an exchange of passengers between the two railroads.

The plan so far described contemplates a measure of traffic requirements of the future. It must anticipate a method of construction that will give the least amount of inconvenience to the traveling public and reduce to a minimum construction hazards and danger to life and limb. It must premise no delay to train movements. During the rush hours of commuting service there is a tube train movement under the bridge every 30 seconds. There are minimum railway clearances and the space available for plant development and all other construction activities is considerably restricted. All the limitations previously mentioned, with the exception of flood conditions, are found in this work.

It will be recalled that the boulevard curved through an angle of 77 deg. as it left the south abutment of the old bridge. Since this turn will always be the point of greatest traffic regulation, the neck of the bottle, so to speak, it was imperative to reduce the angle of this turn as much as possible in fixing the position of the new structure.

This angle of curvature was reduced 10 deg. and considerably improved without materially affecting the turn at the other end of the structure. It was possible with this alignment to construct five main arch ribs of the north span and six of the south span beginning with the westerly face of the bridge together with all the approach work adjacent thereto, and leave the old bridge in the meantime in position to carry the boulevard traffic. The upper view of Fig. 19 shows this part of the structure almost completed. The concrete is being conveyed by a system of chutes, arranged to cover the entire area of the structure, leading from a central tower 180 ft. in height. Because of the limitations to space available for plant and equipment due to railroad track requirement, the mixing plant including storage bins of 100-cu. yd. capacity, was located advantageously adjacent to the center pier in a 12-ft. space between arch ribs No. 6 and No. 7. In this position it is immediately adjacent to the old bridge on the one side and the completed portion of the new bridge on its other side. The top of the bins is placed at a level with and is an extension of the bridge floor so that the materials hauled by trucks may be dumped directly into bins, first conveyed over the old bridge and later over the portion of the new bridge which when completed will also receive the traffic of the old bridge. The old bridge



ABOVE: FIG. A. CENTER PIER CONSTRUCTION SHOWING FLOOR SLAB TO EXPANSION JOINT.

LEFT: FIG. B. SHOWING LIMITED SPACE BETWEEN TRACKS FOR FABRICATING STEEL ARCH CENTERING.

BELOW: FIG. C SHOWING METHOD OF ROLLING THREE SETS OF ARCH CENTERING SIMULTANEOUSLY BY HAND WINCH.

BELOW: FIG. D. SHOWING UNDER SIDE OF ARCH RIBS WITH CENTERING APPROACHING SECOND POSITION.

Fig. 18.

FIG. A. BIRDSEYE VIEW HUDSON COUNTY BOULEVARD BRIDGE PLAZA SHOWING SOUTHBOUND ROADWAY AND SIDEWALK ALMOST COMPLETED TO RECEIVE TRAFFIC BEFORE OLD BRIDGE IS DISMANTLED.

FIG. B. VIEW OF WEST FACE HUDSON COUNTY BOULEVARD BRIDGE PLAZA SHOWING STEEL ARCH CENTERING IN PLACE OVER PENNSYLVANIA R.R. ON LEFT, AND HUDSON & MANHATTAN R.R. ON RIGHT

FIG. 19.

will then have outlived its usefulness and will be dismantled to enable the completion of the final portion of the new structure. The small portion of the new bridge floor occupied by storage bins will remain to be completed as the last concrete operation after the distribution tower and bins have been removed.

The design and construction of the main arch ribs, were other important considerations. The ribs were proportioned to give the necessary train operating clearance over the tracks anticipating the use of structural steel arch centering in their construction. The lower view of Fig. 19 and all the views of Fig. 18, illustrate this feature. A three-hinged arch centering was designed especially for this work. The top chord conforms to the curve of the intrados of the finished concrete arch ribs. There are three sets of arch centering for each span and since there are thirteen concrete ribs to be formed the reuse operations reduced the cost of centering to a comparatively small figure. To move the steel arch centers they are lowered a sufficient amount to clear the underside of the finished concrete arch by striking a system of wedges used in jacking them to the correct elevation. A nest of rollers replaces the wedges and the entire structural steel unit consisting of three sets of ribs is rolled along to the next position. A hand winch supplies the motive power. During the moving horizontal tie rods counterbalance the horizontal arch thrust due to its own weight and facilitates the sliding by relieving the pressure against the sliding skids arranged along the face of abutment and piers. These skids were freely greased to further reduce frictional resistance. It requires less than one day to do the actual moving.

It was no little problem to make the initial erection of the steel arch centering over the moving trains of both railroads. They were assembled on the ground in halves, each half weighing 12 tons. Exactly one hour between 3 and 4 a. m. was allowed between the less frequent train movement of that hour for raising and connecting these heavy frames in their final position. This was accomplished on schedule time without mishap by two large cranes.

A careful provision was made in the pouring of the arch ribs to compensate for the deflection of the centering. The computed theoretical deflection due to maximum condition of loading in pouring of the concrete was approximately ⅞-in. In erection, the centering was cambered 2 in. at the crown to allow for this deflection and for additional deflection due to compression of timber thrust blocks and settlement at the supports. The actual total deflection measured varied from 1¾ to 2⅛ in. The arch ribs were divided into five voussoir blocks, one at the crown and two on either side. Similar blocks of either side were poured simultaneously to give symmetrical loading of the centering. In addition a key block 4 ft. in width at the haunches or skewbacks was provided and left unpoured until the five blocks composing the rib had been poured. The load of these five blocks gave full deflection of the centering, and since they were not joined with abutments or pier by reason of the 4-ft. key, they were free

to move with the deflection. The arch ring bars were lapped in the key to give further unrestrained movement. Two or three days after the main blocks were poured the arch was completed by filling the key sections. If the concreting of the ribs had been started at the skewbacks and carried in continuous operation to the crown, the deflection of the centering would cause serious cracking of the concrete somewhere in the haunch sections. It was interesting to watch the movement due to the deflection of the centering as it was translated and measured at the top of bars formed in the 4-ft. key section.

A considerable amount of traprock was excavated adjacent to the tracks, the old bridge and construction operation, the dynamiting for which was carried on with consummate skill by forces of the contractor. Not a single accident marred this hazardous work. A peculiar rock formation was encountered. Over the easterly portion of the structure the rock outcropped for full depth of the railroad depression, but fell away suddenly to great depth below the track level within the footing area. The entire north abutment is founded on rock as is also the rib shafts of the center pier with the exception of the last four westerly ribs. Only the four easterly rib shafts of the south abutment will be founded on rock. Elsewhere a very compact gravel was encountered and the ribs are founded there on a continuous reinforced-footing slab.

A of Fig. 18 is a view of the center pier construction showing the floor slab in cantilever to form the expansion joint, a detail of which is shown in Fig. 10. A similar joint is provided adjacent to the abutments. In addition there are two construction joints at the junction of the slab and column floor construction with slab construction supported by crown section of the arch. The center pier section was built up by constructing a small portion of the arch ribs with it. This section of the floor was used advantageously as a working platform in carrying on the construction of the arch ribs and the floor slab over them.

Question Box.

1. Given a sudden cold snap in the course of concrete construction, with a lapse in cold weather precaution, or a suspicion in the mind of the engineer in charge as to the possible inadequacy of covers or heating, what is to be the basis of subsequent judgment as to when forms shall be stripped?

Mr. Wason. L. C. Wason (*By Letter*).—This company has been through several such experiences. In our experience one freezing and staying frozen will do the concrete little or no damage; or one freezing and thawing out will not seriously impair the quality of the concrete. Alternate thawing and freezing will ruin it.

We have solved the question by heating the frozen concrete under proper conditions of protection by canvas or otherwise, trying to see that the heat on the ceiling is not more than 100 deg. and at the floor line is 40 deg., or above freezing in any event. After the concrete has properly thawed out it is kept heated for some time until it is proper to draw the forms, namely, from 10 days to 15 according to the care and degree of heat which has been applied after the concrete has been thawed. Under such circumstances we have avoided any ultimate damage to the concrete and have saved concrete which at first appeared to be in a hopelessly bad condition.

Mr. McMillan. F. R. McMillan.—I realize that my answer may not be very helpful. Having tested a good many grades of concrete, grades arising from many different causes, I look for a resemblance in a sample or a slab to some specimen I have broken. Knowing at what strength such specimen broke, I am able to form some conclusion as to what the concrete contains. I believe that such visual examination with hammer or chisel offers a means for anyone who has had such experience to form some conclusion as to the quality of the concrete. Where this experience is lacking I do not see how the method can be very helpful.

Mr. Lord. A. R. Lord.—As I see this question, it involves pretty nearly every building which is built in seven out of twelve months in this climate. This is a question that arises 100 per cent of the time on winter concrete in building operations in Chicago or in similar latitudes. The question sets up a case in which a "sudden cold snap" raises a doubt. Ordinary late fall and early spring weather are as treacherous in this respect as any cold snap for the latter brings its own warning which the former sneaks in with only mildly uncomfortable days to conceal its desperate curing conditions for concrete. Concrete requires both moisture and warmth for curing—and both simultaneously. The absence of either in a satisfactory degree is cause for precautions to be taken. In Chicago, for instance, assuming that a 1 : 6 concrete under ordinary job conditions

gives a 2,000 lb. concrete at 28 days in July and August, the probable strength in other months on the basis of probable temperature and assuming an adequate supply of moisture would be:

	Probably 28 days strength (no heating)		
Months Considered	Mean of 54 Years	Warmest Years	Coldest Years
June and September	1,800	1,900	1,700
May and October	1,500	1,750	1,350
April	1,300	1,500	1,100
November and March	1,050	1,250	850
December, January and February..	700	1,100	

If forms are taken out in less than 28 days as is commonly done even in winter, the strength will be reduced below the above figures. If a floor slab has a cold basement under it, with poor air circulation, its strength may be greatly reduced. This table indicates the need of artificial heating in Chicago from November to March in years of normal temperatures.

The use of a concrete with about 4-in. slump, put in hot and kept at 65 deg. to 70 deg. F. for from three to seven days, is good insurance against collapse when forms are removed.

But the question assumes that there is doubt as to the safety of removing the forms and asks how the issue may be safely decided. I believe it can be safely decided—in fact experience shows that it is commonly so decided—but only by those with first-hand acquaintance with concrete on the job. An old superintendent rarely makes a mistake, although he frequently flirts desperately with disaster—and once in a while one gets thrown for a fall. I believe the best *jury* to decide this issue is made up of one (1) part old superintendent and one (1) part engineer trained in testing concrete. Assuming that the man who asks this question has responsibility in the matter (and he had better not reach for any if it does not belong to him) I would advise him to get his "jury" and proceed as follows:

(1) Be sure that the concrete in question is not frozen. Test pieces from several locations near a radiator. Put in salamanders for one day if in doubt—it will help to dry the concrete out and increase its strength in any case.

(2) Examine the fine and coarse aggregate to see if the materials and the proportions used in the work are reasonably good.

(3) Strip lintel beams, hatches and columns and break off small pieces, testing their cross-breaking strength and appearance of the fracture. Test mortar in these pieces with penknife.

(4) Make the usual crude tests by striking a sharp blow with a hammer (examine indentation and listen to "ring") and by driving in 8-d. nails. These tests are worthless unless repeated many times, over the area to be tested, but under such repetition become quite reliable.

(5) If the jury agrees, take its verdict. If they disagree, take several good-sized pieces of concrete and test for crushing strength.

The standards of judgment on such field tests are based solely on experience. Well-cured concrete has a distinctly different look than green concrete, readily recognized by an engineer with testing experience, and also by many field men. Good concrete can be broken in the fingers; but not easily. Good concrete rings under a hammer and shows only slight indentation. You can drive an 8-d. nail into 2,000-lb. concrete, but not easily, nor will every attempt finally be successful. If you can easily drive in a nail almost anywhere, do not remove any forms until the concrete has had several days of 65 to 70 deg. F. temperature with an ample moisture supply except for the last 24 hours when the slab should be dried out.

If the construction above is not able to carry itself and imposes a load far beyond the designed capacity of the floor in question, such construction must receive the same attention.

J. C. GRADY (*By Letter*).—The question we will assume includes the removal of shores, because the time when the sheeting is removed is of minor importance. Not only the shores in the story which has suddenly been subjected to freezing temperatures, but also the shores under the floors below, which may have been concreted during cold but not freezing temperatures. Provided the concrete has not frozen or alternately frozen and thawed, the question can be readily answered, but if the concrete has frozen the situation then required the exercise of extreme care and judgment. Mr. Grady.

To determine whether concrete is frozen, break off a piece and take it into a warm room, do not place this test piece directly on top of stove or in extremely high temperature, as the effect is very misleading. "Sounding" the concrete with a hammer is not a satisfactory method.

Unusual precautions are necessary when concrete in structural members is frozen because when the concrete has thawed and a temperature of 70 deg. is established this concrete will gain strength very slowly. (See *Proceedings*, A. C. I., 1916, page 241.) This temperature of 70 deg. should be maintained for two weeks after concrete has thawed and the concrete should be kept damp to assist in the gain of strength. Before subjecting concrete which has been frozen to heavy loads, it is desirable to apply a test load or cut sample from same for test.

If the concrete has not frozen a judgment can be based on a study of the temperatures to which it has been subjected. The Portland Cement Association in its pamphlet called "Concrete in Cold Weather" gives a table showing the daily gain in strength of concrete at the age of one to twenty-one days at temperatures from 40 to 70 deg. From this table you will note the great necessity of applying heat immediately when the concrete is placed. On the first day the difference in gain in strength between 70 and 40 deg. is 145 lb. per sq. in., while on the sixth day the difference is only 16 lb., and the twentieth day only 6 lb.

It should be further noted that the tests reported in the A. C. I. *Proceedings*, 1916, show that at temperatures below freezing the gain in strength of concrete which had been heated to 60 deg. for five days are very small. Therefore, in cold climates two stories should be kept enclosed and heated.

If the temperature on the first day was below 40 deg. but above freezing, it would be well to heat for two days longer than is required by the table, to give a strength of 500 lb.

Any floor which has been subjected to low temperatures before the concrete was five days old, should be carefully watched when the shores are finally removed from under it. It must be kept in mind that this floor may then have to carry some increment of load from the floors above which are still shored.

Mr. Colburn. D. S. COLBURN.—Mr. Grady referred to a table that the Portland Cement Association has issued on concreting in cold weather. I believe that Mr. Hart would like to say a word about that table.

Mr. Hart. W. E. HART.—It has been found this year that a number of the calculations in that table are not absolutely accurate. So far as the daily increments are concerned, we have no hesitancy in standing behind it. However, if you add up your total number of pounds for the 28 days, you will find that you will only get about 1,500-lb. concrete where you should have 2,000 lb. The daily increase is satisfactory and the part referred to, I believe, is all right.

2. What has experience taught in construction and in concrete products manufacture as to variations in hardening properties of standard portland cements at various temperatures and under special conditions of placing and curing? Are standard cements relatively uniform under ordinary conditions? Under unusual conditions?

Mr. Bates. P. H. BATES.—You will have noted from the question that the wording "standard portland cement" is used. I did believe for a while that we had a standard cement, but recently certain sections of the country have prepared standards of their own. In addition of course there still remains the standard known as American Standard No. 1 of the American Engineering Standards Committee, which is identical with that of the American Society for Testing Materials and the Federal Government, and finally we still have, according to advertising literature, "the standard by which all others are measured."

Portland cement itself is just as heterogeneous as is this apparent condition in regard to our standard. It is composed of three distinct compounds—tricalcium silicate, tricalcium aluminate, and dicalcium silicate, to say nothing of a number of minor constituents. I want to show through a few slides and curves that portland cement of average composition will contain these constituents in different amounts and that these constituents in the presence of moisture react to different degrees, and then finally ask, "Can you expect uniform results from such a heterogeneous material, or must we not always expect and consider each cement a distinct problem in itself?"

In the three figures presented on pp. 615-7 is under consideration, first, the different constituents; second, different cements; third, how the different constituents are affected by the presence of gypsum, and fourth, how fineness affects the absorption of water. In all the curves "A" refers to the material so ground that it will pass a No. 325 sieve. "B" refers to the material which has passed a No. 200 sieve but retained on a No. 325. The letter "C" indicates that the material is a commercial cement. The letter "K" indicates that the material is ground clinker corresponding to the cement of the same number. No extended discussion of these figures is needed. They are self-explanatory, and as indicated before they show distinctly that the different compounds present in cement will absorb mois-

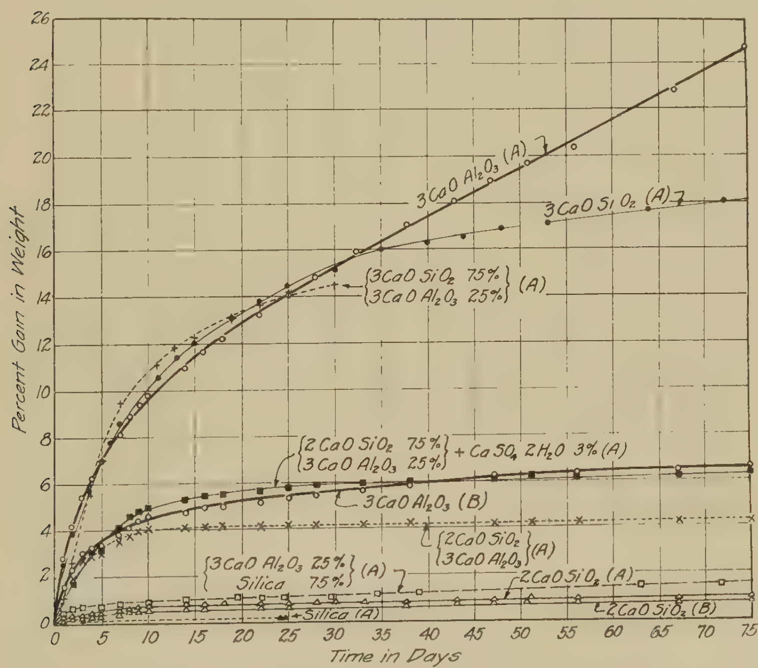


FIG. 1.—PER CENT GAIN IN WEIGHT OF CONSTITUENTS OF PORTLAND CEMENT KEPT AT 96 PER CENT RELATIVE HUMIDITY AS A FUNCTION OF TIME.

(A) Material which passed through No. 325 sieve.
(B) Material which passed through a No. 200 sieve but was retained on a No. 325 sieve.

ture when placed in an atmosphere at a relative humidity of 96 per cent at quite different rates and amounts, that the presence of gypsum materially alters this rate and amount, and that the degree of fineness again materially alters this problem. It is also quite evident that the different commercial cements are quite different in their ability to take up water.

With such data before us, can we expect our so-called standard cements to give us uniform results under unusual conditions, and possibly under usual conditions?

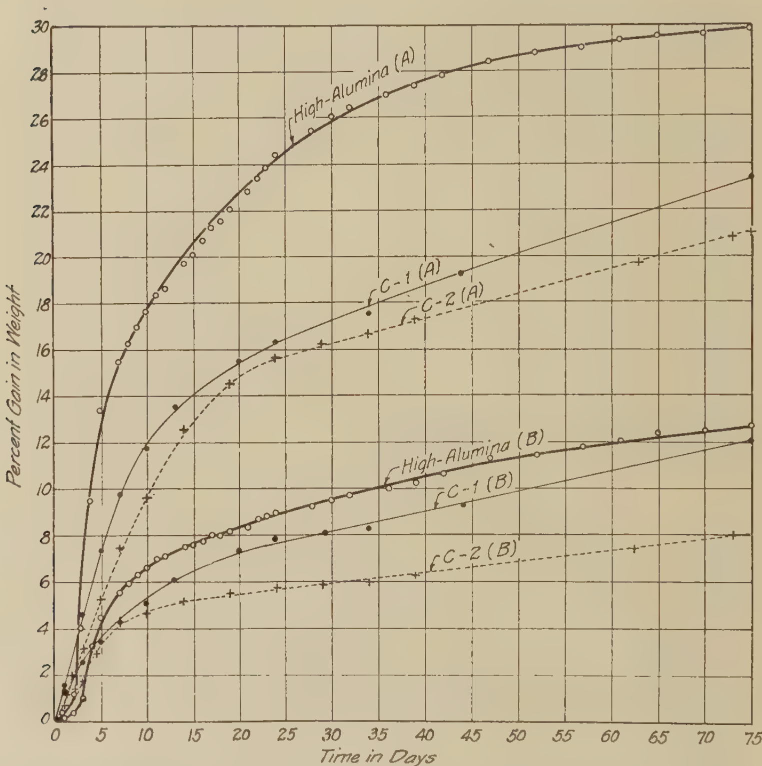


FIG. 2.—PER CENT GAIN IN WEIGHT OF HIGH ALUMINA AND PORTLAND CEMENTS KEPT AT 96 PER CENT RELATIVE HUMIDITY AS A FUNCTION OF TIME.

(A) Material which passed through No. 325 sieve.
(B) Material which passed through No. 200 sieve but which was retained on a No. 325 sieve.

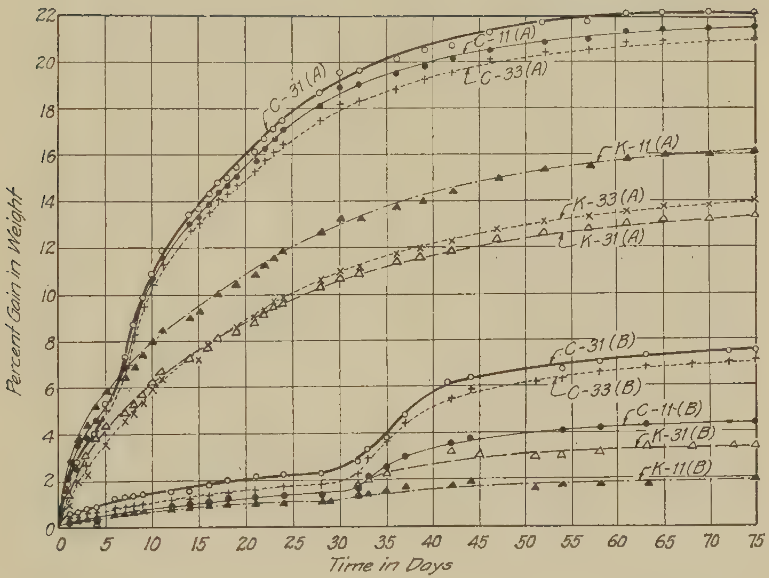


FIG. 3.—PER CENT GAIN IN WEIGHT OF SOME PORTLAND CEMENTS AND THEIR CORRESPONDING CLINKERS KEPT AT 96 PER CENT RELATIVE HUMIDITY AS A FUNCTION OF TIME.

(C) Cements; (K) Clinkers.
(A) Material which passed through No. 325 sieve.
(B) Material which passed through No. 200 sieve but which was retained on a No. 325 sieve.

Mr. Hollister. S. C. HOLLISTER.—There is in existence a table showing a number of tests made by the municipal testing laboratory in Philadelphia. There is a considerable number of brands of cement represented. It gives the number of samples and the number of barrels per sample represented. There is a fair uniformity in the values as to time of set, and as to strength, with the one exception of Atlas Lumnite, which does not fall in the group of cement coming under the standard test for portland cement. I want to point out distinctly the fair uniformity in strength and time of initial and final setting. The temperatures of the laboratory range from around 70 deg. up to 75 deg. or 76 deg. Tests conducted at different room temperatures, revealed the difference in time of final set due to difference in temperature. At 70 deg. to 75 deg. F., the final set is fairly well grouped for the particular brands here shown. When the temperature is lower during the conduct of the test there is a continually widening zone between the cement represented by the upper curve and that represented by the lower curve until, at a temperature of 32 deg. F. there is over 100 per cent difference in time of set.

It is not intended that these data be taken as a standard value for cements in general or even for any cement in particular, but it is intended that they be taken as indicative of the wide range between the various brands of cement. Considering tensile strength at 24 hours of three brands of cement common in the eastern market, one notices that at 32 deg. F. one cement had no strength, one had very little and another was up about 180 lb. to the square inch. It would be helpful to know by what means the standard tests would show this tendency of the curves to diverge at the lower temperatures. To carry this into the field, it has a direct effect on the time of stripping surface forms. If the cement sets up slowly under the particular temperature conditions, the forms must be left on a longer time or there is danger of pulling the cement mortar surface of the wall off with the form. Obviously the result is that a builder will select his brands of cement knowing that they have particular characteristics at lower temperatures. Even under the 60 deg. temperature condition he will undertake to select those brands that will give him as high a strength at this particular temperature condition at as early a date as possible.

Mr. Hutchinson. G. W. HUTCHINSON.—It seems to me that we can think over what Mr. Bates has said in regard to the lack of uniformity and possibly supplement his discussion with physical tests. I agree with Mr. Bates that we do not have anything but general standards for portland cement. The A. S. T. M. standards are about the only ones we know, but I question whether they will satisfy in many cases. It is the question whether we consider the average of two, three or four briquets of cement from a single sample; the range of cement from a given mill throughout a month; or cement from several mills throughout the year. We also have to consider the structure in which the cement is used. We usually design our concrete for 28 day strength and assume that the 28-day strength is satisfactory.

In highway construction however it develops somewhat into a rate of hardening rather than into the actual strength of the concrete at this period. While with the North Carolina Highway Commission, we decided that the present standard cement test was of but little value in this respect. We undertook to investigate cement from about 18 different mills. At 28 days those cements varied the 1 : 2 : 4 concrete strength from 1,500 to 4,000 lbs. per sq. in., yet at later ages some cements which might have indicated 1,500 lb. at 28 days showed as much strength as those which ran 3,000 lb. So it is not always economic to select by the early strength, especially in highway work where you approach a need for ultimate hardening instead of high early strength.

A good many cements are rejected unfairly. We had one experience where cement did not harden satisfactorily for sixteen days, yet at the age of three months, we found that cores from this section gave us the strongest concrete we had in the state. I think it develops into a matter of getting more definite means of determining the value of each cement which should extend beyond such tests as are made at the age of 7 or 28 days, and should include the rate of hardening. In other words, let that influence the selection of cement as far as the construction is concerned. We cannot depend on the present specifications to detect such values in our cements.

Mr. Ahlers. JOHN G. AHLERS.—I believe there has been a feeling right along that we were getting on dangerous ground when we began discussing the strength of cement. But I believe also it is a question that is going to be of increasing importance as we get more and more familiar with controlling our concrete with a strength specification. For instance, when we compare our test results with such as Professor Abrams has from his laboratory, where he has mixed many different brands of cement together to get an average cement. I do not believe that the tension test of the A. S. T. M. we are now using for determining the quality of our cement is going to be satisfactory to us in the future; I believe that we should parallel that with compression strength tests of the cement. The necessity for such was brought out in connection with some tests we made when tracing back the variation from batch to batch in some concrete specimens where from the various batches fortunately we had saved and tested some samples of cement in compression. There were three bags in each batch and samples were taken before it was dumped in the mixer. We found curious analogy between the strength of the cement-mortar specimens and of the different concrete specimens, so that the difference in strength of the concrete was directly traceable to the cement which was all out of the same car. I hope that the cement manufacturers will co-operate to investigate cement more from this angle.

Mr. Kinney. W. M. KINNEY.—I would like to raise one point in the interest of our old friend cement. Aren't we dealing with a material which, from its very inception, is going to be somewhat non-uniform? Just what uniformity are we after? We are handling a material which is sold for ten

to eleven per ton. We have got to have and must expect some non-uniformity. Just what are we after here?

Mr. Lindau. A. E. LINDAU.—Is there anyone who is willing to answer that question? As nearly as I can gather we are after a fine, high-grade cement.

Mr. Ashton. ERNEST ASHTON.—Isn't that what we have? I am not representing the cement manufacturers. I have listened to several discussions with regard to the non-uniformity of cement, and I might say, as a cement manufacturer, that the most pertinent thing we have to consider is that very question. We have struggled for a number of years trying to find a method of test that will reveal this uniformity. We went through the specification process of tensile strength and then switched over to the testing of cement in compression. A survey of the papers presented at the meeting this year, indicate we should now change back as tensional value and not compressional values are needed. We have not reached the point yet where we know what are the merits of the material itself.

Let me say this with reference to variation with respect to brands amounting to 40, 50, or 65 per cent. We took an individual sample a year ago and sent it out in behalf of Committee C-1. We made as homogeneous a sample as possible and sent it to 19 of the most representative laboratories in the United States. We did not get any 40 or 50 per cent variation in test result, we got 100 per cent variation. We are more serious about this than you are; this is our life blood; this means everything to us, it is an ordinary passing event to you; we have not reached the point yet where we can satisfy ourselves as to what is the method that will give us the interpretation of the cementitious value of the material.

If you have got something to offer us we are ready 100 per cent to fall in line, but what is it? Let us reverse this question. Where does this non-uniformity lie? Does it lie in the material or in the matter of test?

3. By what methods can high early strengths be obtained with standard portland cement?

Mr. Chubb. JOSEPH H. CHUBB.—The marked effect of water on the strength of both neat cement and concrete is clearly shown in the following tabulation of results of tests at our research laboratory:

Mix	Water Cement Ratio	Slump Inches	Compressive Strength Pounds Per Sq. In. 1 Day	2 Days	3 Days	7 Days	28 Days
(Neat	0.14	Dry Tamped	2800				
Cement)	0.23	" "	5400 (Maximum strength)				
"	0.28	" "	4100				
"	0.35	Normal (Consistency)	2500				
"	0.39		2100				
"	0.65	Grout	550				
1–5	1.15	7–9	170	510	720	1110	2580
1–5	0.93	½–1	370	860	1120	1940	3970
1–3	0.78	7–9	660	1310	1940	2780	5030
1–3	0.63	½–1	1040	2070	2550	3660	5610

The strength of mortars and concretes depends on the water-cement ratio so long as the mix is workable. Mortar is sand held together with glue made of cement and water. The stronger the cement water glue the stronger the mortar. Concrete is coarse aggregate held together by mortar and any desired strength within a wide range may be obtained by varying the water-cement ratio.

Common practice in construction work is to specify mix and to obtain increased slump or workability by adding water, thereby sacrificing strength. The right practice should be to specify water-cement ratio and to increase slump or workability by cutting down volume of aggregate thereby maintaining strength and at the same time obtaining the desired workability.

M. M. UPSON.—As a user of cement who occasionally has to face the problem of early-setting, particularly in the manufacture of concrete products, I consider it important that the characteristics of the different cements be known. I do not agree with the intimation made by one of the previous speakers that it is dangerous to open up this avenue of discussion. Most of the members have a very high regard for the accomplishment of the cement manufacturers. Almost no bad cement is delivered; but it is idle for us to close our eyes to the fact that there is a wide variation in the characteristics of the cements which comes from various factories. Some cements show a high early strength; others give very low strengths up to ten or fifteen days; and some do not show their full strength before the end of a sixty or ninety-day period. However, it is pleasing to state that most all cements eventually do show a satisfactory compression strength if proper mixtures are used. **Mr. Upson.**

The important point that I desire to make is that with a knowledge of cement characteristics it is possible for the user to employ this information to his economic benefit.

In producing concrete products, the size of the kiln is determined by the time required to develop a strength sufficient for handling. If that period can be reduced 50 per cent by the use of a quick-setting cement, the kiln cost is reduced by half, and an equal saving is made in the cost of heating and humidifying.

I have knowledge of tests made in plants, where there has been a variation in the early compressive strengths of different brands of cement of as much as 400 per cent. This does not mean that the product showing the lower early set did not eventually attain almost as great strength as those which show the high early strengths.

It is obvious that the cement showing the early-set is worth much more to the user in this particular service than that showing a slow set. For this reason it is important that full information on the characteristics of cement be made available.

ROBERT SETH LINDSTROM.—Calcium chloride has been mentioned as obtaining an early set in concrete. A lot of literature has been published on that subject. Lately we have heard a good deal about it. There is no **Mr. Lindstrom**

question in my mind but that calcium chloride does make early set, but we want to be very cautious how we use it. I do not think anybody should use calcium chloride in a concrete building where there is steel reinforcement.

4: Under what conditions is the use of retempered concrete permissible or of advantage?

Mr. Abrams PROF. DUFF A. ABRAMS.—I think that the most striking thing which I could say on this subject is that, to my mind at least, the old impressions concerning this subject are largely erroneous. Our specifications generally stated and the older text-books and discussions on this subject almost always asserted that concrete should not, under any conditions, be held longer than 30 min. before being placed. That is generally accepted as an axiom in concrete work. The facts are that concrete would be considerably improved if it were possible to hold it for a period of say 30 min. to three hours, as the case might be, depending upon conditions, before it is put in place. In other words, I believe we would get very much better concrete, assuming that it is always plastic. I believe that concrete would be greatly improved and not injured, by holding it.

We have made some tests along this line and such tests have been made by others. So far as I know in every case this new impression has been borne out. Concrete can be held, and, by slight re-mixing, with ordinary cement and materials in the usual proportions, for periods of three or four hours and still show an actual increase in strength over the similar concrete molded immediately.

That brings up another question which I think should be mentioned. We make a time of setting according to the specifications by using a cement mixture of normal consistency, a very dry, stiff mixture of neat cement. The mixture that we actually use in our concrete has no resemblance to the fixed mixture on which we make our time of setting test. The time of setting as determined on neat cement has only the slightest relation to what actually happened in the concrete because the conditions are so different that there is no basis for comparison. In our concrete we are using probably three times as much water as we are using in our normal consistency test. That is the reason we get such slight correlation between the time of setting and the actual hardening of the concrete.

I believe there is good reason for retempering with concrete. There are some places where it is almost impossible to get satisfactory results without retempering. About the only satisfactory method I know of is to clean the surface and make a fairly stiff mixture and let it stand for a certain period of time before it is put in place. What happens during this period? I am not certain that I know everything that happens, but I think that the most important thing that happens is that the water in the mixture is absorbed by the cement and the aggregate has a tendency to reduce the final shrinkage of the mass which ordinarily would cause trouble in forming a patch, and in other cases, such as floor construction, where a minimum of shrinkage is very desirable.

P. H. BATES.—In this question of the use of retempered cements, I might cite some observations which I personally made a couple of years ago. A house was being built back of my home in Washington, and on reaching home one night I was surprised to find that the plasterer had left his mortar board filled with the stucco which he had been applying and that furthermore the mortar box was also full of the mixed stucco, which incidentally was made of a white portland cement. Being curious to know whether he would discard this amount of mixed stucco I went around in the morning and waited until work was started. There was no hesitancy at all on the part of the workman to regage the entire lot of left-over stucco. It mixed up very nicely into a good fat stucco and was put on the house. It is still on and has weathered two winters excellently. Mr. Bates.

From this you might deduce that that particular brand of cement, if retempered, was all right, or if it was put on a house on the street immediately back of where I live in Washington it would be all right. However, as a matter of fact, I believe that the conditions under which you can use retempered cement would be those under which you wanted to get away from the job as quickly as you could and never see it thereafter.

In some of the previous discussion regarding this question some statements have been made in regard to the relation between time of set and strength of neat cement and the time of set and strength of concrete. I have never seen any convincing data showing that there is a definite relation between the two. In fact, all data so far presented has thoroughly convinced me that the strength of neat cement is no measure whatsoever of the quality of that cement in concrete.

5. How can you apply the principles of accurate quality control to the making and placing of concrete on small jobs?

F. R. MCMILLAN.—We have on this leaflet information which I used successfully in controlling a small job. A blueprint from which this leaflet was prepared was handed to the contractor with no other instructions. After reading this and with almost no question or comment the work proceeded. The work was carried out exactly as I would have done it myself, with no argument or complaint at any time and almost without comment. Mr. McMillan.

In this case the instructions were quickly understood and easily carried out. There is no reason why any man with high school training, in fact, any foreman who can read, cannot understand the method and apply it.

One point to which attention should be called is the basis of the curve at the top of the leaflet. This curve is not the one from Bulletin 1, Structural Materials Research Laboratory, represented by the equation $S = \frac{14000}{7^x}$ frequently used in the design of mixtures. This is the one from Bulletin 9 ($S = \frac{14000}{9^x}$) recommended for concrete where the placing and curing conditions are not so well controlled. In these equations, S represents the compressive strength at 28 days when cured in moist atmosphere and x (an exponent), the water-ratio by volume.

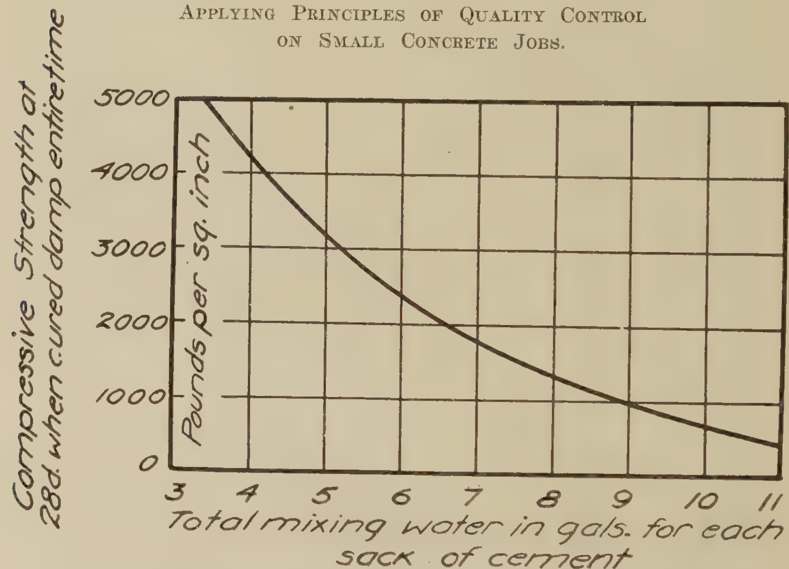
The diagram, of course, is in terms of total mixing water, which means water carried by the aggregate, plus that added at the mixer.

The three notes beneath the diagram call attention to the amount of moisture carried by the aggregate. These are sufficiently close for small jobs when working with the more conservative curve just indicated.

To get concrete of any desired strength, select from the above curve the amount of mixing water to use for that strength.

Then mix sand and stone as desired until the required consistency or workability is obtained, using all the time the correct ratio of water to

APPLYING PRINCIPLES OF QUALITY CONTROL ON SMALL CONCRETE JOBS.



(Above strength curve from Fig. 1, Bulletin No. 9, Structural Materials Research Laboratory, by Abrams and Walker.)

Note 1: By "total mixing water" is meant water in the sand and stone plus that added at the mixer.

Note 2: Very wet sand will contain from ¾ to 1 gal. per cu. ft.
Moderately wet sand will contain ½ gal. per cu. ft.
Moist sand will contain about ¼ gal. per cu. ft.
Moist gravel or crushed rock will contain about ¼ gal. per cu. ft.

Note 3: The coarser the sand the less water it will carry.

cement as found from the curve. When sand or coarse aggregate contains moisture, the water to be added at the mixer will be the amount shown by the curve less that carried by the sand or coarse aggregate as indicated in Note 2.

The more aggregate that can be added to a given quantity of cement and water, the more economical the concrete, the limit being that the mix must not be too dry or harsh to place readily.

Illustration: If concrete having 1800 lb. per sq. in. is desired, the curve shows that the total water required is 7 gal. per sack of cement.

Suppose moderately wet sand and dry crushed limestone were to be used. A first batch should be made up, say, using 1 bag cement, 2 cu. ft. sand and enough coarse aggregate to produce required consistency. Two cu. ft. of sand as per Note 2 would carry a total of 1 gal. of water. Therefore, it would be necessary at the mixer to add $7 - 1 = 6$ gal. If it were found desirable to use more or less sand to get the desired workability, say $2\frac{1}{2}$ cu. ft. of sand, then the correction would be $1\frac{1}{4}$ gal. and water to add at the mixer $= 7 - 1\frac{1}{4} = 5\frac{3}{4}$ gal. for each bag of cement.

T. P. WATSON (*By Letter*).—The first requisite for manufacturing quality concrete on any job regardless of its size is a man with sufficient knowledge and experience to oversee and direct the operation. **Mr. Watson**

The second essential is an arbitrary specification for the quality of the concrete, which in lieu of a more definite and as yet undeveloped requirement, should be expressed in compressive strength of pounds per square inch at an age of 28 days. This strength specification should be amplified with an exact number of gallons of water to be used to each sack of portland cement and under no conditions should this ratio of water to cement be exceeded. It also should be distinctly understood that the moisture in the aggregates is to be considered as a part of the permissible number of gallons of water to be used.

The third requirement is the use of evenly graded qualified aggregates. Evenly graded aggregates may be defined as aggregates of definite minimum and maximum sizes and a practical average uniformity of the in between sizes.

It will be assumed that the fundamental requirements of concrete manufacture will be observed, i. e. the use of a mixer in good condition, some method for the accurate volumetric measurements of the aggregates separately, and that all concrete will be mixed at least one and one-half minutes and longer if possible.

With the foregoing conditions fulfilled the application of the principles of accurate quality control in the field manufacture of concrete is a comparatively simple procedure.

The following outline is a practical method of field control requiring a minimum mental and physical effort.

Predetermine arbitrary proportions by calculation or trial, according to the workability required, of one sack of cement to cubic feet of *saturated* fine aggregate and cubic feet of loose coarse aggregate.

The reason for suggesting a proportion of saturated fine aggregate is that saturated fine aggregate of the same grading and from the same source of supply will have a constant volume and therefore a constant dry net volume and weight.

To be brief, if we will assume that arbitrary proportions of cement, saturated fine aggregate and loose coarse aggregate have been calculated, and that the net dry weight of one-half cubic foot of saturated fine aggregate and the net dry weight of one-half cubic foot of loose coarse aggregate have been determined, the explanation of the procedure to be followed

will enable you to use the same principles in the design of mixtures by the trial method.

The equipment required is two metal cylindrical receptacles (capacity one-half cubic foot each; inside height ten inches; and a diameter of ten and thirty-one-sixty-fourths inches) and a double beam scale of a capacity of one hundred pounds.

Our problem is to determine the relation of the volume of loose fine aggregate to be placed in the mixture to the volume of saturated fine aggregate of the proportions, the weight of the water in a unit volume of the loose fine aggregate and the weight of the water in a unit volume of loose coarse aggregate.

For convenience referring to the two metal receptacles as "A" and "B" the determinations are made as follows:

Fill "A" about one-third full with water.

Fill "B" level full with the loose fine aggregate to be used in the mixture.

Weigh "B" and obtain the net weight of the fine aggregate of the loose sample.

Carefully pour the contents of "B" into "A."

Measure from the top of "A" to the surface of the saturated fine aggregate. This measurement enables you to determine the relation of a volume of loose fine aggregate to a volume of saturated fine aggregate. Also knowing the volume of the saturated fine aggregate of the sample you can readily determine the net dry weight of the sample, which deducted from the weight of the loose sample, gives you the weight of the water in a volume of the loose fine aggregates.

Next obtain the average net weights of six determinations of a receptacle level full with the loose coarse aggregates. The difference between this average weight and the known weight of a volume of dry loose coarse aggregate is the weight of the water in the loose coarse aggregate to be used in the mixture.

We now have all the information required and on the basis of the proportions used, the moisture in the aggregates is deducted from the specified gallons of water and the gallons of water added accordingly.

The use of a diagram and correlative information similar to Fig. 1 will reduce the mathematical calculations necessary to a minimum.

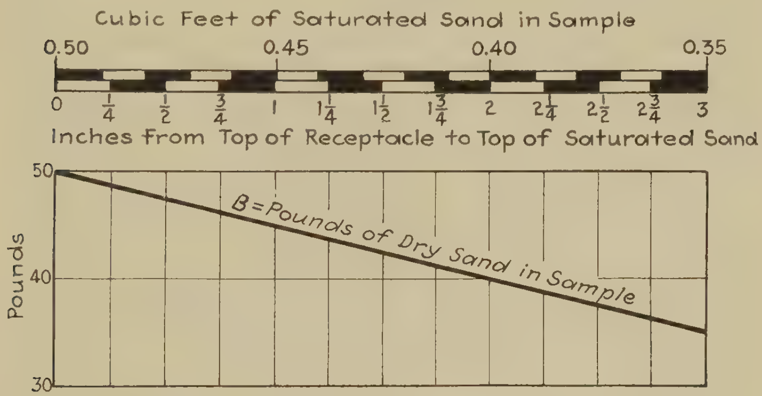
If the trial method of proportioning is used and you find that in order to obtain the desired workability more water is required than specified, the additional water should be added together with a proportionate number of pounds of cement based on the proportion of the number of gallons of water specified to a sack of cement (94 lb.).

If on the other hand the trial method of proportioning disclosed that the water-cement ratio was lower than specified, aggregates both fine and coarse should be added according to judgment, taking into account the moisture they contain, until the desired workability is obtained without exceeding the specified water-cement ratio.

Having obtained the desired workability, it will be necessary to convert the proportions for subsequent batches to a unit of cement to avoid the use of parts of a sack of cement. For example suppose that a satisfactory trial batch contained 338.4 lb. of cement (3.6 cu. ft.)—8 cu. ft. of loose fine aggregate and 13 cu. ft. of loose coarse aggregate. Dividing these quantities of aggregates by 3.6 we obtain a proportion of 1 sack of cement —2.22 cu. ft. of loose fine aggregate —3.61 cu. ft. of loose coarse aggregate. The conversion of the proportion of loose sand to its saturated volume can readily be determined.

In conclusion I would draw your attention to the necessity of care in making your determinations and to standardize your methods so as to exclude the personal error as much as possible.

This applies to both aggregates and more particularly to the method of placing the sample of loose fine aggregate in the receptacle so as to



approach as nearly as practical the same compactness as it will be measured in whatever volumetric batching device you may be using to charge the concrete mixer. As an illustration we have found that by dropping the loose sand from a scoop at a fixed vertical height into the cylindrical receptacle, we approached the same compactness as that of the sand in the batcher. The fixed vertical height referred to was the mean of the vertical fall from the bottom of the bins to the bottom of the batcher.

If wheelbarrows are used or volumetric measurements are made other than with batchers, the placing of the cylindrical receptacle in the wheelbarrow or other measuring device and the filling of the receptacle by the same method as is generally employed by shoveling or otherwise will obtain excellent results.

I know that sounds rather complicated, but the actual physical workings are very simple. I will attempt to show you what I mean by using two glasses. Assume that these two glasses had a volume of ½ cu. ft.; you would fill this glass one-third full of water, fill this container with a

loose sand and you would weigh that. That is the weight of the least sand with the receptacle full. Pour that sand into that water; that sand will not be to the top, it will be lower, and if that is 10 in. high, it is a very simple proposition to stick a rule in and find the volume. That net volume in there is constant and you can make a chart. All one has to be able to do is to read and then pick the weight off and make the determination in two minutes. Taking this weight and deducting the weight of the dry sample gives the moisture in the sand.

I would like to say that I think that Mr. McMillan's idea of judging three grades of sand hardly practicable. We have found that the variation of the moisture in sand is very marked; it has run from 1 to 10 lb. in a cubic foot of sand.

Mr. McMillan. F. R. McMILLAN.—I do not wish to argue the matter, but we had our job about half done in the time it has taken Mr. Watson to read his paper. The proceeding is really very simple. With half a gallon per sack margin between this and the more exact water-ratio curve, great refinement is not called for. On this little job, we put a pail of water and half a sack of cement into the mixer, then shoveled in sand and crushed limestone, until the desired consistency was obtained. The number of shovelfuls of each was counted. These amounts were used in subsequent batches until some change was desired. It was not only very simple, but very satisfactory.

Mr. Young. R. B. YOUNG.—For several years engineers of the Hydro-Electric Power Commission have been doing just about what Mr. McMillan has proposed here, and it works all right on the job. If a man is so dumb that he cannot understand that, he should not be in charge of the job and if he is so stubborn that he won't follow them, he also should not be in charge of the job, and the remedy is obvious. We have had no trouble in applying a scheme similar to this to small jobs of 400 yd. or so. We do not do it exactly this way, but we do it in a way that is fundamentally the same.

Mr. Grady. J. C. GRADY.—If on a small job aggregate of good quality and proper size is selected and the job organization is careful to mix the concrete as dry as practicable, the results will be a great improvement over what is now obtained on many jobs where sloppy concrete is allowed. A further advancement is to adopt on the small job the water-cement ratio method explained elsewhere in this volume.

6. In earth-filled highway arches, what is the most suitable waterproofing practice?

Mr. Cohen. A. BURTON COHEN.—You will recall yesterday in my paper that I emphasized the fact that correlating the waterproofing problem were the problems in design. I emphasized the importance of considering waterproofing before design was completed. I emphasized also the proper termination of the waterproofing, otherwise the money is poorly spent and protection is not properly made. I have here a cross-section of the parapet wall of the Corning Bridge which was shown yesterday. I may explain the method of construction, the method of waterproofing, etc. The arched ring is 42 ft. wide and in its construction we brought it up a distance of about

a foot above the main portion, in order to increase the strength of the ring, give it the proper strength to support this sidewalk construction which has this cantilever design. That also is a very good construction from the standpoint of drainage, because if anything should happen to the waterproofing at this point, you have a little dike that will carry the water down and it will not percolate through that construction job. That was the first operation.

The next operation was to carry the parapet wall up and leave a recess in it for the cantilever construction. This is a counterfort construction and the reinforcing bars were carried up and were used for the reinforcing in the cantilever. After that work was completed, that second operation of concrete, the waterproofing was applied which consisted of two two-ply saturated cloth, three applications of asphalt. The waterproofing was carried up to the top of this construction, flashed around the counterfort. The next operation then was to build the sidewalk overhang with the beam, the curve acting as a flashing to hold the waterproofing in place. It also increased the rigidity of the cantilever construction, because the curve is monolithic with the cantilever beam. Where we placed the expansion joint, we divided the counterfort into two parts and had separate designs, beam designs, for each one. The waterproofing was splashed. In the second stage of the arch construction recesses were left in the parapet wall and the reinforcing rod projected out to take the cantilever beam. This waterproofing is brought up to the top of the wall and flashed at its top.

We have protected the waterproofing in the barrel of the arch 1½ in. of concrete. On the sidewall we plastered with about a ½-in. thickness of cement mortar, first spraying it with a broom, in order to get a little body and then building up in three courses the plaster until we reached about ½ in. covering. That is just to protect the waterproofing from the air, give it a greater life, and, as the fill is applied to the back of the arch as it is deposited in the back of the arch, not to injure the waterproofing, that is the biggest function of that protection.

7. *What methods should be used for economical but thorough placing of concrete in deep beams having a large number of closely spaced reinforcing bars?*

T. P. WATSON (*By Letter*).—The first requirement for placing concrete in heavily reinforced members is that the uniformity and workability of the concrete is such as to permit the puddling of each batch to insure a homogeneous mass. **Mr. Watson.**

Uniformity is maintained by the constant use of identical proportions of cement, properly graded aggregates and water.

Workability is that property of concrete so essential to the manufacture of a durable product and the property which is practically impossible to cover in a lucid definition. As workability applies to reinforced concrete, it may be partially defined as the proper plastic flow perceptible to the experienced concrete craftsman.

Economical workability is obtained by the use of proportions of fine and coarse aggregates of such percentages so that when combined dry and rodded they make a unit of maximum density. These proportions can be determined by the Abrams theory of "fineness modulus."

Maximum workability can only be obtained by study and experimentation with the available aggregates or in other words maximum workability is obtained by having a sufficient quantity of mortar to prevent segregation and at the same time permit a plastic flow.

The economical and thorough placing of concrete in reinforced members requires special arrangements although in principle they should be the same for placing concrete either plain or reinforced.

The proper placing of concrete requires that the batch be delivered to the form by some method that will permit its placement as near as practical to its final position.

This can be accomplished by short chutes attached to movable hoppers or other practical means.

The placing of concrete should be preceded by the placing of a batch of mortar with the same water-cement ratio as is used in the concrete. This mortar should be distributed from one end of the form toward the opposite end and in no case should it be above the level of the bottom lines of the bottom row of bars.

After having placed the batch of mortar concreting should be commenced by starting at the same end of the form at which the batch of mortar was placed. The concrete should be deposited to such a depth, usually 12 to 15 inches, that the act of puddling the concrete will push the mortar originally deposited ahead of the concrete.

This procedure should be continued until the approximate middle fifth of the length of the member is reached at which time the same procedure should be started at the end of the member opposite the end where the concreting operation was commenced.

By continuing the placing as outlined the two concretings will meet and at that time there will be a level layer of concrete throughout the lower section of the form.

Subsequent layers should be placed in the same way, alternating the commencing of the layers from each end of the form, but omitting the batches of mortar.

This method permits the widest possible diffusion of laitance or excessive mixing water if any and insures bearings and ends of the best quality concrete which are especially desirable in structures exposed to outside climatic conditions.

Methods of puddling concrete will be governed by local conditions and size of the members. As general practice a sufficient number of men, equipped with long handled wooden paddles and hoes with perforated blades, should be employed to insure puddling that will produce a homogeneous mass throughout.

The danger sign of concreting is the formation of laitance or puddles of water in the form.

When either appear it is notice that the concrete is being drowned and immediate steps should be taken to correct this condition.

Usually this can be accomplished by placing a few batches with a little less water but be sure that they are not too dry. If too dry they will do more harm than if they were too wet.

With certain types of aggregates, especially the use of a coarse graded sand, the elimination of laitance and puddles of water in the forms is rather difficult.

In such cases the remedy will be to increase the quantity of sand and cement and decrease the coarse aggregate proportion, but as a general principle economical aggregate proportions should be predetermined which will avoid lack of workability usually known as harsh concrete and there is no defensible reason why harsh mixtures should be tolerated in reinforced-concrete work.

J. C. GRADY.—The best and most economical method which I have found by experience to be very effective, is to omit all or part of the coarse aggregate from the first batch placed. In a beam as described in the question, I would omit all of the coarse aggregate in the first batch and dump this mortar in the beam between the support and the one-third point and follow immediately on top of same with the regular proportioned concrete, spading and jarring the reinforcement with a blade tamper. The mortar which has been placed in the beam will furnish a bed for the concrete to drop into and will be displaced by the concrete and slide along sluggishly, filling in thoroughly between and around the reinforcement. The amount of mortar necessary to properly start the work as described is entirely dependent on size of beam, amount of reinforcement, etc., and can only be determined by experience and judgment. Size of aggregate and consistency of the mix must be suitable for the work on hand. **Mr. Grady.**

9. How shall we make plaster bond to relatively smooth concrete surfaces?

L. C. WASON.—This company has also had quite a bit of experience of this kind. In the application of hard plaster to smooth concrete surfaces and in co-operation with the master plasterer where in some cases the work has scaled off we have come to the conviction that this is due to the skill in tempering the mortar on the stage by the actual mechanic who is placing it on the ceiling or wall of smooth concrete. This we proved in one important instance by the fact that three plasterers were working on the same stage and we could trace out the work of one plasterer which failed and the other two were perfect long after the building was finished, and by watching their method and by talking with the men we discovered that the man whose work failed did not temper his work right and after he was instructed by his employer to temper his mortar to get the right results he gave good results thereafter. It is our opinion, therefore, that this problem is entirely in the hands of the master plasterer. **Mr. Wason.**

Mr. Floyd. G. F. FLOYD.—We have applied gypsum plaster to comparatively smooth concrete surfaces of buildings as ceilings, walls and columns. There was no other treatment of these surfaces than the careful application of the gypsum bond plaster to them. Dressed lumber was used in the forms for the concrete. After the forms had been removed the concrete surfaces received no further treatment. A manufactured gypsum bond plaster was carefully applied before the brown mortar and white plaster was applied.

The buildings where the above method was used have been standing for four or five years, and we have had no complaints that the plaster had loosened up in any way.

There was a committee of New York City plasterers appointed by the school board to investigate the trouble that had been experienced on account of the white plaster falling off some of the city schools. This committee was asked to investigate the cause of the trouble and make recommendations to overcome it. After a careful investigation the committee recommended that metal lath be placed on all concrete surfaces before plaster was applied.

A member of our Company, the Turner Construction Co., had occasion to talk to the chairman of the committee sometime after the recommendation had been made. The chairman stated that in every case where the gypsum plaster had loosened from the concrete they found that the trouble was due to one of two causes. In some cases the bond plaster had been carelessly applied and often omitted entirely, and in others had been applied too far in advance of the brown mortar. Where too long a time had been allowed to elapse between the placing of the bond plaster and the brown mortar the bond plaster had thoroughly dried out and the surface became glazed. In this case the brown mortar did not adhere to the bond coat but fell off leaving only the bond plaster on the concrete.

The chairman stated that they had recommended the placing of wire lath on concrete surfaces before applying gypsum plaster because they felt that the committee should be very conservative in recommending a method to be used in school work where the plastering contracts were let under close competitive bidding. He felt, however, that no trouble would be experienced if gypsum plaster was applied to comparatively smooth concrete surfaces provided a gypsum bond plaster was first carefully used and the brown mortar placed as soon as the bond had sufficiently dried.

Mr. Foster. C. B. FOSTER—I applied about 1000 yds. of plaster to concrete and observed it during a period of five years. In that time there were three different places where some of the plaster came loose. I used a bond coat and found that the only reason it came loose was on account of smoke from the salamander or something on the ceiling that had not been removed. Some five or six years ago I tried it again from plaster direct to a factory building where there was considerable vibration, and less than a week ago I had occasion to visit that building and there is not the slightest

crack in the ceiling. My experience has been very successful in applying plaster to concrete surfaces.

M. M. UPSON.—What is your bond coat? Mr. Upson.

MR. FOSTER.—I used what the gypsum plaster people put out as a bond coat; I do not know of what it consists. Mr. Foster.

M. M. BERNIER.—A bond coat is a neat gypsum without any sand. The gypsum is applied as thin as possible so as to not have too much strain on the ceiling. Then that is followed by the bond coat with neat gypsum. We depend on the suction of the concrete with the smooth white coat, which should be done in an hour or two. Mr. Bernier.

VIRGIL L. JOHNSON.—I have taken a view of this subject from the standpoint of the architect and engineer, because he first wants to be sure that the work is going to stand. In Philadelphia we build schoolhouses of the skeleton concrete or factory type. We do not use flat slabs owing to the shape of the room, but we do fill the panel with a joist and slab construction and underneath that joist construction place wire lath which is plastered. As the columns are spaced about 15 ft. 2 in. on centers, you will see that there is in an average classroom two large girders which occur across the classroom overhead. That construction is exposed; that is the girders are not plastered; we never have any trouble with the plaster on the wire lath ceilings. There is however a possibility of some trouble with the soffit of the large girder, which is 16 in. wide. Mr. Johnson.

10. What experiences have indicated that cracks developing in concrete can be healed?

This is a highly important subject to discuss, in fact to those architects and engineers who have had and are having experience with "falling plaster" or cracked ceilings (and there are many) it becomes a vital question. To the responsible contractor, to the owner and to the manufacturer of portland cement products, it is an all important question because a satisfactory ceiling without cracks must be obtained at any cost. As an example, suppose we consider a public building or a public school, and concrete constructed school houses are now being built all over our country in every town or city.

In this particular school which we take as an example, it is two or three or more stories in height. Of course it is fireproof and panic proof. Exits and fire towers are so arranged that the children in the building can be marched out within ten minutes in perfect order and with no fatality.

But with all this care, the building is not accident proof because there is no telling when or at what time a section of the plaster work may separate itself from the concrete beam or girder over a crowded classroom and fall upon some child.

A school building or any public building built of concrete may be completed as prescribed by the contract and built according to carefully written specification, the contractor may be paid in full and time may pass for six or eight years and still the plaster is good but perhaps by the ninth year a section of plaster in a particular beam over a particular

classroom separates from the concrete and falls with fatal results. It is found upon examination that this particular beam had a "wind" in its side or a "belly" in the soffit and the plasterer, wishing to make the work good "trued up" his plastering so that in some places it is an inch or more in thickness. The piece that has fallen off is 1¼ in thickness, 20 in. in length and 8 or 9 in. wide, weighing 12 to 15 lb.

We can draw our own conclusions as to the importance of this question of bond. No damage money which a board of education or a city provides, can ever compensate for a fatality of this kind.

In the city of Philadelphia last year, 240 children were killed by accidents of various kinds. In the most cases, they were in the street and often the child's fault. To the public in general, it was often passed on by a short paragraph, in each case, each case in its turn was read with a comment that it was too bad, etc. But let one of those 240 cases occur by a falling piece of plaster or concrete in a school and we would have had great head lines in the newspapers calling attention to "Dishonest Contractor, Faulty Construction" or "Careless Inspection" and the board of education would have been censured severely.

Now, as to a remedy or rather a preventive against the occurrence of these accidents, let us look into the subjects of an adequate bond between plaster and concrete.

This question may at this point present itself: Why do we use plaster on a concrete surface? There are two very good answers to this:

First—We wish an even surface for decoration and for the elimination of "dust catching" places which the usual concrete surface presents.

Second—We must have a plastered surface to assist the acoustics of a room—that is very necessary in the classroom or auditorium.

The use of a "treated" surface or a "hacked" surface on concrete is not always sufficient because lime mortar does not adhere to a concrete surface with the same tenacity that it will to brick or hollow tile.

If the plaster is ½ in. or more in thickness, it has a tendency by its own weight to drop off and the thicker the plaster, the more dangerous it becomes.

The only satisfactory bond that can be used with any assurance of safety is something on the order of mechanical bond, a wire lath or expanded metal. This wire lath or expanded metal should be carefully secured to the beam or girder and preferably the plaster should be two-coat work, first a brown coat and then the white finished coat.

A very satisfactory and inexpensive bond can be secured through the use of what is commercially know as chicken wire. The chicken wire is galvanized, of small gage, can easily be stretched over the concrete girder and is secured to it by wire or clips that have been previously embedded in the concrete.

The use of chicken wire permits of a very thin coating of plaster about ¼ in. thick, will stand as long as the concrete will stand. There seems to be no other way to bond plastered concrete successfully than by

the use of the mechanical bond such as has been described. The securing of the wire mesh to the girder may present some difficulty and I offer as a suggestion the use of a certain lath grip, an invention that has recently come to my attention.

J. E. FREEMAN.—The question of sufficient bond is not only influenced by methods of application of plaster to concrete and by preparation of the concrete surfaces beforehand, but time is an important factor in the problem. Mr. Freeman.

Some of the influences which develop subsequent detachment of plaster applied directly to concrete surfaces are slow acting. Plaster may be sound and hard yet its bond with the surface gradually affected so that detachment occurs suddenly after a period of several years when the adhesion to the surface is no longer sufficient to support the weight of the plaster. It has been said by experts on plaster, that a critical time for plaster on concrete is during a period of 5 to 10 years after the plaster has been applied, and there are examples with which the writer is familiar where detachment of plaster applied directly to concrete has occurred after periods up to 20 years. The plasterer may well feel relieved when the period of one year or 18 months' guarantee, which the arichtect requests of him, is passed without trouble developing, but this is no criterion that a slow action is not taking place which will cause detachment later on.

Hacking the surface of the concrete to roughen it relates to only one angle of the problem—that of obtaining some sort of mechanical key for the plaster. It does not overcome the influence of irregular and excessive suction, expansion and contraction or chemical action, any one of which, as experience has shown, can weaken the bond and develop detachment of plaster applied directly to the concrete. These influences can only be combatted effectively by sealing off the surface of the concrete from actual contact with the plaster, which also protects the plaster and surface decorations from stain or greater injury by moisture or efflorescence from the structural body.

A method that accomplishes this result successfully and is being extensively used is described in a publication of the Gypsum Industries Association, dealing, among other things, with specifications for plastering on concrete surfaces (See Gypsum Plasters, general instructions and specifications—Gypsum Industries Association, Chicago, 1925). On page 18, in connection with the treatment or conditioning of concrete surfaces prior to plastering, under the heading "Waterproofing Materials" appears the following statements:

"A commonly used process applied over concrete surfaces, prior to plastering, consists of driving into and over the concrete surface, by force, a series of layers of asphalt cement into which, while still plastic, there is afterward driven, also by force, a course rough grit. This process, which also waterproofs the structural surface, provides uniform suction and a positive mechanical key or clinch for the plaster.

"The process referred to will provide all that is necessary for the

permanent adhesion of plaster to concrete surfaces, and is of special value since it permits of the application of the plaster when the concrete surface would ordinarily be regarded as too wet or too dry. Gypsum plaster, because of its insulating and quick setting properties, is the most economical and efficient material to use in connection with this process."

A particularly severe test of the effectiveness of the process, developed through its use on the D. L. & W. R. R. station at South Orange, New Jersey, is referred to in discussions of reports of Committee C-3 in the *Proceedings* of this Institute for 1922 and 1923.

In the application of plaster to concrete surfaces, there should be developed a factor of safety, just as there is in the design of the structural details of a building, and methods should be adopted for preparing the surface and for applying plaster thereto that will develop and insure a factor of safety against plaster detachment.

10. What experiences have indicated that cracks developing in concrete can be healed?

Mr. Abrams. DUFF A. ABRAMS.—This question of the healing of concrete is not altogether new; it has a good many interesting phases. There may be some angles to it that we have not as yet explored. I have observed, however, in a number of cases that concrete which has been ruptured at an early period will, if left exposed to the elements, gradually heal up and become even stronger than it was originally. The most striking evidence of that kind was some tests we made at the Lewis Institute a few months ago. Eight years previously we had tested a good many concrete cylinders to failure at the end of 28 days. These cylinders were placed in the yard, and some years later we decided to put them in the machine again and re-load them, and they not only took as much load as they had originally taken, but we got values all the way from 167 to 379 per cent of the original 28-day strength of this concrete. You might inquire what happened. I think what happened is this. Those small fissures in the concrete which were opened up at the time of the original test are actually healed up, welded together, by the subsequent depositing of the soluble materials from the cement and aggregate. It is actually a healing process and the wounds have grown up, the concrete proceeds to gain in strength much as it would if it had not been subjected to its original load.

That, I say, is probably the most striking and interesting example that has come to my attention from a purely laboratory standpoint. However, a few years ago, earlier still, I had occasion to make a test on a reinforced-concrete highway bridge in southern Illinois built by the state highway department for testing purposes. Before my own connection with the job it had been loaded when the bridge was about three or four months old and some very decided cracks had occurred in the end of the girders, not one crack but both ends of both girders were cracked on both sides, and in many instances showed a number of cracks, the characteristic diagonal cracks you see in girders of that kind that are overloaded. My first impression was that this bridge was in a rather hazardous condition. We

spent a good deal of time putting props under it in case it failed. We actually put nine to thirteen times the original load on that bridge and some of those cracks never did open, but new cracks opened alongside of them, showing that the original cracks had entirely grown up and were as strong or stronger than the original concrete. Those two illustrations, I think, bear evidence of the fact that this healing feature is a real factor and may be of interest in other connections.

J. J. EARLEY.—In the spring of 1914 the Bureau of Standards at Washington appointed an advisory committee and began an earnest investigation of Stucco. I had the good fortune to be appointed to this Committee and became very interested and active in the work. I began to observe more closely and with better understanding the characteristics of stucco. My attention seemed to fix itself particularly on cracks and I never lost an opportunity to study them. **Mr. Earley.**

This led to the discovery that the cracks in stucco sometimes heal, and further investigation disclosed that the cracks in concrete pavements and in concrete structures sometimes heal. In healing the crack gradually fills with a cementing material (carbonate of lime) which completely closes it and finally raises a small ridge not unlike the ridge raised by scar-tissue. In fact one receives a very definite impression that the crack has healed.

In 1917 in connection with the construction of an entrance on the west side of Meridian Hill Park in Washington we made a very long balustrade, each panel of which consisted in a foot rail, a row of balusters and a hand rail. The work was described in the Institute's *Proceedings* for 1918 but it may not be amiss to recall that the surface of the concrete was treated before it was thoroughly set by brushing it with steel wire brushes until the aggregate was exposed uniformly over the surface. It was necessary to apply this treatment to all four sides of the hand rail because the bottom of it could be seen between the balusters. Therefore, it was necessary to turn the rail over once before the cement had hardened enough to resist the action of the wire brush.

Several of these rails were broken in the turning and were discarded before my memory of what I had seen cracks in stucco do suggested that the breaks might be healed.

Purely as an experiment a broken rail was bedded down straight and true in sand. The fracture was covered with sand and then with burlap. Twice a day this bandage was saturated with boiling water until the fracture was seen to be thoroughly knit together and almost invisible. The section of this rail was 7 by 14 in. The cast was then allowed to dry and, although it was 14 ft. long, it was handled and set in the usual way. It has remained sound until now.

After this experiment established the fact that broken concrete will heal, I have, whenever the occasion has arisen, deliberately used the process with uniform success. However, I have always applied the process to new concrete; that is, to concrete from which the original mixing water has

not evaporated. Consequently, I do not know to what degree control may be exercised over the healing of old concrete which has thoroughly dried. But I feel sure, after certain observations in the field, that the power to heal remains in old concrete.

I do not know how long a period of time is necessary for the healing process. In the studio the cast is usually wetted once or twice a day until the crack is filled, then it is allowed to dry. I assume that the amount of cement in the concrete, the temperature, and the circumstances in which the cement is acted on first by the water and then by the air have a great deal to do with the process of healing and the time required for it. I assume that the action of water and air in healing concrete is similar to the action of the same elements in forming stalactities.

I cannot recommend a general application of this principle because I cannot indicate a technique for its application. All that I can do, therefore, is to record in the *Proceedings* of the Institute that certain breaks in concrete have healed, and that with concrete of such uniformity as that used in my studio the healing is under some degree of control. And, furthermore, that the technique will probably be built upon an operation which will consist of alternately flushing and draining the fissure in a suitable temperature.

Mr. Hollister. S. C. HOLLISTER.—This* shows a six span structure consisting of arches of 80 ft. and span rise of about 14 ft. The arch on the right had been concreted from the right and to about a third of the span lifted. There was no concrete in place over the center pier. It was desired then to concrete the portion over the center pier. The centering of the left arch was in place to the extent shown in this picture at the time the incident I shall describe occurred. The centering and steel were all in place on the right arch. Concreting was carried on simultaneously on the third end portion of the left span and the third end portion of the right span over the center pier. The concreting was done in the summer and, as sometimes happens, the cement set a little more rapidly than normal.

The centering went down about an inch. Because the concrete had set a little faster than was usual under the temperature conditions and because of certain interruptions in the proper progress in the concreting operation, the concrete immediately over the pier which was 4 ft. deep in that particular place, was cracked all the way through to the full width of the bridge immediately over the face of the pier. That crack was on the righ hand side of the center pier. The crack opened about ⅛ in. on the top face. I was not present at the time the crack occurred, but I was called by telephone and asked what should be done.

I profited very much by a previous discussion with Professor Abrams and Mr. Earley and their experience in the healing of concrete. Here was a structure in concrete at a very early age, and it looked as though the opportunity to try the healing was better on this occasion than it would be at almost any other time. The crack formed was not a fresh fracture, such as would be found in the test of a structural member in the labora-

*View not printed here.

tory, but it was rather a quake. It had occurred just at the time when the concrete was beginning to harden.

The width of the barrel is 26 ft. The thickness directly over the edge of the pier is 4 ft., the span is 80 ft., and the rise 14 ft. We immediately put on sand to the depth of about a foot; we put dikes at both ends of the pier; we stuffed up the drains in the low portion where the two barrels come together, so that they could not release any moisture. We covered the pile of sand over with bagging and the bagging and the sand were kept continually wet for four days.

Part of the time the water was retained actually in excess by means of the dikes. Bagging was hung over the side edge of the barrel to protect the drying out of the crack where it occurred down the side of the barrel. Since the crack had opened up so much it was not possible for one cement surface to jump over to the other cement surface, and since the crack was not incipient, as has been described in the other cases of healing, it was necessary to introduce something to serve as a filler, and this sand with the excess water carrying it down slowly as far as it would go, served that purpose.

Examination four days later showed that only about one-third of the crack was visible. A week following that, examination disclosed that the entire crack had disappeared. It is not now possible to detect the position of that crack.

From that time since the concreting operation then proceeded as before, as though nothing had happened. The centering was struck from the arch 28 days from the time the crack was discovered. It is an earth-filled arch.

Reports of Committees of
The American Concrete Institute

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Report of Committee E-6, on Destructive Agents and Protective Treatments.

During the current year, the committee has continued its studies of deterioration of concrete. It has had for its consideration a number of reports by members of the committee and others. So far as the material has been released for publication, it is included in this report.

The most interesting case of concrete disintegration which has been brought to the attention of the committee during the year is an example

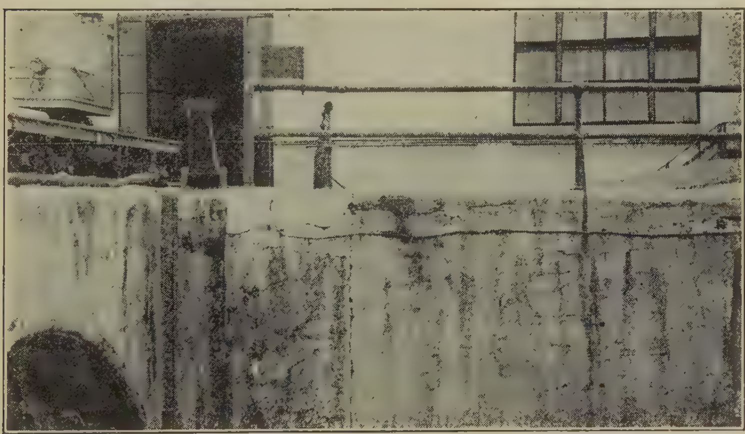

FIG. 1.—ARCH RIB AND WALL SHOWING SURFACE DISINTEGRATION.

This picture shows a portion of an arch rib over a spillway section and a connecting wall after 14 years of service. In Fig. 2 is shown the depth to which similar disintegration in another arch of this same structure had proceeded during a period of 8 years. The indications all point to over-wet mixes as the principal cause of the porous concrete resulting in the disintegration.

interesting both in itself and because of the light that it throws on some of the questions discussed in the report of the committee presented to the Institute at the 1925 convention.

Fig. 1 shows a portion of an arch over a spillway dam and a portion of the forebay wall connecting with it. The structure is located in one of the Central Western states and has been exposed to about the same climatic

conditions as are found in Omaha, Nebraska. It was built of limestone concrete about 14 years before this picture was taken. The surface markings indicating disintegration, which are prominently seen on the forebay wall, are characteristic of the face of the arch ribs. This is not so clearly shown in the figure owing to the fact that the portions of the rib nearest the camera are not in focus.

FIG. 2.—SECTION OF ARCH RIB AT CROWN SHOWING DEPTH OF DETERIORATION.

This section, which was cut after the structure had been in service 8 years, was made to determine how far deterioration had progressed. A block 4 ft. long, 2 ft. wide, and 3 ft. thick was removed from the upstream side by drilling. White dotted line indicates the approximate limit of the softened concrete. Inside this line concrete is solid and unaffected in any way. At top edge solid concrete was found 7 in. from face. First drill hole, which was about 10 in. back from face, broke through when 9 in. from under surface of arch. Drill holes next to traveler rail (visible in the picture) broke through when 3 in. from under surface of arch. Concrete back of traveler rail unaffected except for the skin coating.

The records show that the concrete was placed rather wet throughout this structure. This is established both by photographs of the construction operations and by the statements of individuals who were present during the construction.

Fig. 2 shows a section cut from one of the spillway arches when the structure was about eight years old. The section was cut for the purpose of finding to what depth the disintegration had progressed, for at that time the arches showed the same evidence of disintegration as is shown in Fig. 1

except that the cracking and markings had not progressed to quite the same degree.

The figure shows the extent of disintegration very clearly. The concrete beyond the line marking the limit of disintegrated material was found to be perfectly solid and unaffected.

The information conveyed by the photograph of Fig. 2 shows rather conclusively that the disintegration is the result of weathering from the surface in the manner pointed out in the 1925 report of this committee. The primary cause of the disintegration is the porous concrete. In this case, the evidence indicates, porosity was largely the result of using very wet mixtures. There is no evidence from the study of this or other portions of the structure that disintegration of the aggregate itself had any important part in the deterioration of the concrete.

Another group of structures has been brought to the attention of the committee by an engineer who has had an opportunity during the past year to visit and visually examine a number of large concrete works located in a region where the climatic conditions are rather severe as regards long winters and low temperatures. He states that, on the whole, more fairly good concrete than bad is encountered, but that real high-grade concrete is quite rare and structures without some defects are very rare. As a general comment on the group of structures, he points out that in a majority of cases in which the concrete is bad, it has been due to aggregates of inferior quality as to grading having been used in proportions definitely fixed in the contract before the quality of aggregates to be used was known. It was his opinion that it would have been possible to have made good concrete with many of these aggregates by the use of richer mixes. In most cases, the structures were built from concrete using a pit-run aggregate without varying the cement-ratio, regardless of the ratio of sand and gravel which varied from ½ to 1½ during the work on a single structure. Aggregates containing silt and organic matter were used in some instances, also the concrete was mixed with an excess of water. Three specific cases are cited:

(1) A large dam about 15 years in service has shown marked disintegration of the spillway. This is being repaired by a coating of gunite. In this case the statement was made by the chief engineer of the company responsible for the construction of the dam, that the chief causes for the disintegration were too lean a mixture and excess of water. Pit-run sand and crushed stone of an igneous formation had been used.

Another defect of this dam is at the joints in the piers which were apparently made with insufficient care. Water follows these joints from the back to the front where spalling takes place and efflorescence is deposited. Otherwise these piers are in good condition.

(2) A very heavy wall subjected only to the percolation of ground water and surface rains has shown signs of disintegration after about eight years in service. In this case, the distintegration is due to too lean a

mixture, excess of water and a poor quality of crushed stone which breaks down when exposed to the atmosphere.

(3) The wharf, about one year in service under the water; the concrete is now eroded to a depth of 1 to 2 ft. back from the face of the wall and that repairs are immediately necessary to save the superstructure. The fine aggregate used in this wall was a fine sand graded from 0 to No. 8 showing a slight color in the sodium hydroxide test and having a strength of 1: 3 mortar about 85 per cent of that shown by standard sand. The coarse aggregate was crushed limestone of good quality and graded from 1 to 2 in. The proportions were about 1: 2: 4. Concrete was deposited under water with the tremie and was mixed with an excess of water. The forms were not watertight. There was considerable movement of the water on the inside of the forms caused by wave action.

This engineer sums up the experiences of his organization with the statement that they have "come to the conclusion that the best and cheapest way for exposed concrete in rivers and lakes is to make an impervious concrete of about 3,000 lb. per sq. in. strength, taking care that the construction joints are made watertight."

In conclusion, the committee desires to restate its findings from the more extensive investigation forming the basis of its 1925 report.*

> "The committee believes that the observed defects were due to porous concrete resulting largely from faulty manipulation of the materials. Permanent structures can be achieved by the use of clean aggregates of durable minerals, a mixture of a fair degree of richness, the use of a puddling consistency and care in placing and curing."

In reporting these findings, the committee desires to point out that they are not intended to cover every possible type of disintegration such as for example, that due to sulphate water. They do apply, however, to a very large percentage of so-called disintegrated concrete. It is also a fair deduction to say that if the principles stated are applied, the resulting structures will be much more resistant to destructive agencies of every kind than where these simple fundamental rules are ignored.

M. M. UPSON, *Chairman*
F. R. MCMILLAN, *Secretary.*

*See 1925 *Proceedings*, American Concrete Institute, pp. 266-280.

Appendix to 1926 report of Committee E-6.

SAND FOR SEA WATER CONCRETE.

By Maxwell M. Upson.

In 1920 a pier was built on the North Atlantic Coast, which was supported by concrete caissons. These caissons were approximately four feet in diameter and varied in length from 40 to 60 ft. They penetrated 30 ft. of water at high tide, and from 10 to 30 ft. of soft bay bottom to hard pan.

These piers were placed by driving a circular steel sheet piling cofferdam, and dredging by means of an orange peel. The footings were tremied, and the columns were poured in the dry by the use of an interior steel form. This method permitted the concrete to set in the dry unaffected by the action of the tide or salt water. During the time of setting, exhaust steam was turned into the annular space between the outside sheeting and the form, for the purpose of hastening the setting of the concrete so that the time of pumping might be reduced.

Because of the wide range of tide the beams connecting the piers are submerged 6 to 8 in. at extreme high tide. During construction the beam concrete was poured and kept in the dry because the maximum tides were not then prevalent.

Within a year of the time of completion of the pier evidences of disintegration were observed. At the end of two years disintegration had progressed to a degree that made it necessary to take steps to repair in order to save the structure. Some of the piers showed no deterioration whatever, while others were affected through their entire cross-section. All disintegration occurred between low and high tides.

This work was done by reputable and experienced contractors and engineers. They have for years carried on concrete work exposed to salt water action. In fact, they have work in this same harbor that shows no evidence of failure although it has been exposed to the action of salt water tide for more than seven years.

Because of the deep interest in this problem a very careful investigation was made of the materials and methods. It was first thought that the destruction was the result of certain chemicals contained in the water, due to the proximity of an oil refinery. Careful sampling and testing of water, however, indicated that the percentage of impurities was not sufficient to warrant the rapid and unusual disintegration.

For the reason that the work was done at a time of great material shortage, the cement was necessarily supplied by the owner; therefore its quality was not checked, although it showed every evidence of being satisfactory and was of a well known and reputable brand.

The gravel and the sand came from a local bank. The gravel in itself is good quality, but the sand is fine and shows evidence of some vegetable matter. At the time this sand was selected there was a question of its suitability. Both the contractor and the engineer recommended the use of

a coarse, sharp, and cleaner sand, but because of the economic saving and for the reason that the 7-day test on the sand showed a tensile strength averaging about 123 per cent of standard Ottawa sand, the owner felt that it should be used on the work. When the mixture showed a deficiency of gravel enough crushed stone was added to approximate a 1: 2: 4 mixture.

An analysis of the fineness of the sand gave the following results:

Analysis No. 1-A		Analysis No. 2-A	
Size of Sieve		Per Cent Finer Than	Per Cent Finer Than
Inches	No.		
0.50	¼	100	100
0.25	5	98	95
0.16	12	85	86
0.0583	20	73	75
0.0335	30	55	57
0.0148	40	43	45
0.0110	50	28	30
0.0055	100	8.2	10.75
0.0030	200	2.2	4.75

It is to be observed from the above that 8.2 per cent is finer than No. 100 sieve, and that the very finest layer from this bank showed 10.75 passing No. 100 sieve.

In looking up authorities it seems to be pretty well demonstrated that notwithstanding the high strength, a fine sand produces a mortar which is easily attacked by the chemical action of sea water. This is discussed by R. Feret* as follows:

> "If a series of —— mortars are subjected to a continuous filtration of sea water, those made from coarse sand remain intact, while decomposition is more and more active for mortars containing more and more fine sand. In practice this latter is the most frequent case, and in fact it has been verified that the destruction of concrete or mortar by sea water has in most cases been due to the use of too fine sands.
>
> "This is a point which cannot be too strongly insisted upon, and experiments show that a rather lean mortar of coarse sand is much preferable to a mortar of fine sand even when a very large quantity of cement is introduced into the latter. Fine sands ought to be banished relentlessly from sea water construction even when the cost of coarse sand is very high. When stone is at hand, an excellent sand can be obtained economically by crushing it."

*See Taylor and Thompson's "Concrete Plain and Reinforced," Third Edition, Page 278.

Tests on this sand indicated that traces of organic matter were present. While the amount was not sufficient to make it objectionable, for ordinary concrete, for sea water work contaminations of this kind should be avoided.

This structure was satisfactorily repaired at a cost which approximates the saving resulting from the use of the finer sand. This was accom-

VIEW OF PIER SHOWING EXTENT OF DISINTEGRATION AND METHOD OF REPAIR.

plished by the use of the cement gun. All materials on the area affected were removed by air tools, and later the surface was sandblasted. As soon as this was accomplished the cement gun was used to build up each member to its original size and form. Great care was used to see that the sand for this service was of a coarse, sharp character, and that the cement met sea water specification.

REPAIR OF FAULTY CONCRETE STRUCTURES.

By B. C. Collier.

The studies of this committee are of considerable interest to the Cement-Gun Co. as the cement gun is used extensively in making repairs to disintegrated structures. The portability of the cement gun and its ability to place concrete in almost inaccessible locations without the use of forms, makes it peculiarly adapted for such repairs. This adaptability has led to its rather wide use for this purpose and has given the writer and his organization a good opportunity to study concrete structures that have proven faulty. A review of our findings may be of interest in the consideration of this subject. In carefully analyzing the reasons for the defects encountered it is found that they come chiefly under three headings:

1.—Excess mixing water.
2.—Improper aggregate.
3.—Improper design.

Concerning the first, the work that has been done within the last few years in water control has focused attention on this important detail and it may be expected that as the knowledge spreads, better practice will result. However, the spread of this knowledge is slow and many contractors still feel impelled, because of the large amount of steel in the bottom of beams, to use excess water in order to fill and surface the bottom of the beam. In some instances where an excessive amount of steel is used, it is almost impossible to surround the steel with material other than mortar.

Concerning the use of improper aggregates, attention should be called to the prevalence of magnesite limestone, particularly in the eastern section of the United States. This slacks rapidly when exposed to air, and is naturally susceptible to slacking if it is in a concrete which is of a porous nature. This action is aggravated in retaining walls or similar structures subjected to seepage and water pressure.

A well-known example of failure due to improper aggregates is that of the Nashville bridges. The stone in these structures was shaley limestone, which turned white and chalky as lime in a distance of two or three inches from the surface. This defect is aggravated from two other sources; namely, fine sand not well graded, which makes porous concrete, and overwatered concrete which is referred to above.

In the neighborhood of Allentown, Pa., where limestone is used as an aggregate, there is evidence in some concrete of a disruption which might be termed "blowing."

The improper design referred to above has largely to do with the placing of the steel. It is not uncommon, even on the part of our best engineers, to space 1-in. or even $1\frac{1}{2}$-in. rods, 3 in. or less on centers, often with

a second layer of rods so placed as to practically cover the opening. This makes a congestion of steel that prevents concrete containing any stone from passing through the intervals and filling the lower portion of the beam. To avoid "honeycombing" in such cases, it is necessary for the contractor to use wet concrete. This results in an exceedingly poor mortar in that portion of the beam which should be densest and most impervious to the action of moisture and exterior destructive agents. Samples are available taken from the bottom of deep girders where not a piece of stone can be found in the slab four inches thick.

It is recommended that engineers avoid the congestion of steel as much as possible, and where for structural reasons this cannot be avoided, it is suggested that the steel be placed directly against the forms and the bottom of the beam be coated with gunite. This will add somewhat to the expense, but will insure a definite coating and proper mix surrounding that portion of the beam which is the most important.

DISCUSSION

Mr. Miner. JOSHUA L. MINER (*By Letter*).—Many failures of concrete, directly or indirectly, are ascribed to the use of excess mixing water. It is generally fair to assume that where badly-proportioned aggregates or fine and dirty sands have been used that an excess of mixing water was also used even though this fact is not stated or not known. Badly-proportioned aggregates result in brash mixtures and in an effort to obtain workability the contractor adds additional mixing water. Fine sands or dirty sands or both require more mixing water than do clean, well-graded sands to obtain workable mixes.

Excess mixing water results in more porous concrete and also concrete of low strengths. Does the effect of the excess mixing water stop here? Is it not possible that the chemical structure of the binding media formed during the process of hydration is affected by the amount of water mixed with the cement? Does the use of excess mixing water result only in a mechanical separation of the individual cementing particles and a mass of less density, or does it in addition result in a binding media that is less resistant to the chemical attack of moisture?

Various distinctive types of concrete failures in both salt and fresh water have been described. Some of these types are found in concrete subjected to either kind of water. One in particular that is common to both is the decomposition of the concrete into a soft pasty material. The binding media or mineral glue holding the aggregates together seems to have disintegrated into a soft sticky material having little if any strength.

It would seem that the low strength of the concrete was not necessarily the cause of this type of failure nor does it appear that porous concrete always fails in this manner. Messrs. Atwood and Johnson in their report on "Marine Piling Investigations," National Research Council, 1924, make the following statement:

"Generally the available evidence seems to indicate that the rapidity of chemical disintegration is roughly proportional to the permeability of the concrete but some structures built with lean mixtures and the consequently comparatively porous concrete are reported where the service has been satisfactory. Such concrete is subject to disintegration by ice in the colder latitudes, but the cases where it has a good service record in warmer waters seem to indicate clearly that some binding media resist chemical attack, even though the water has easy access to the interior of the mass."

Low strengths may be an indication of the use of an excessive amount of mixing water. Porosity also may be an indication of the use of

excessive mixing water. Further, porous concrete undoubtedly will disintegrate more rapidly under the chemical attack of water than will impermeable concrete. However, in such cases it would seem that the failure was not due to a low strength porous concrete but to a relatively poor resistive cementing material.

D. R. Miller, drainage engineer, U. S. Department of Agriculture, in his article, "The Action of Sulphate Water on Concrete," *Public Roads*, October, 1925, states:

"The relative consistency of 1 is as wet as can be used in machine-molded tile, and it has been found that consistencies greatly varying from this are considerably less resistant."

It would appear from the foregoing that Mr. Miller had determined the optimo amount of mixing water which would give the maximum durability for the cements he was using and that an increase in the amount of water resulted in a cementing medium less resistive to the chemical attack of sulphate waters.

B. Jeanneret, in his discussion of Atwood and Johnson's paper "The Disintegration of Cement in Sea Water," Transactions, Am. Soc. C. E., Vol. LXXXVII (1924), states:

"The original chemical composition of cements is no guarantee of their value. It is the chemical combination which takes place in the moist mortar which is the principal factor in their ability to oppose the destructive action of sea water."

In other words, it is the chemical constitution of the set or hardened cement that determines its resistance to the action of sea water.

Apparently the chemical character of the binding media or hardened cement is affected not only by the amount of curing water, but also by the method of curing. Mr. Miller, previously referred to, has reported, "None of the standard portland cement cylinders cured in steam at a temperature of 212 deg. F., regardless of the length of time they were so cured or of other variables, showed any surface action," while "curing in water vapor at temperatures of 155 deg. F. and 100 deg. F. has been of no value." These statements are quoted from his paper on the action of sulphate water on concrete previously mentioned.

Laitance is an example of the effect of a great excess of mixing water. The cement has been drowned, hydration has been carried beyond the point of maximum strength to practically no strength. The material is highly porous and will not withstand weathering.

Available evidence seems to indicate that the chemical structure of hardened or set cement mixed with an excess of water differs from that mixed with an optimo percentage of water. Concrete made with an excess of water appears to be not only weaker and more porous but also less durable.

Mr. Watson. T. P. WATSON (*By Letter*).—Mr. Upson, in the appendix to the 1926 Report of Committee E-6, refers to the mortar produced by using a *fine sand* and quotes R. Feret regarding the chemical action of sea water on mortar made of too *fine sands.* The writer feels that the promiscuous use of the words fine and coarse as referring to sands with no other qualifying sizes of the particles has caused a great deal of confusion and misunderstanding in the minds of concrete users who are eager to put into practice any knowledge which will help the production of better quality concrete.

The fine sand of one section is considered as coarse at another point and practically each locality has its own trade side classifications of sands which adds to the general muddled state of mind.

The writer also takes this opportunity to call attention to the fact that the published data on the prospective merits of different gradings of sand, based on laboratory mortar tests, are very misleading as they apply to guidance in the manufacture of durable concrete.

The natural outcome of the accepted practice of determining the relative qualities of mortar based on a so-called *consistency* is the fixed opinions among investigators and laboratory authorities that the use of the coarser sands will produce the better quality concrete.

The absence of any practical relation of *laboratory consistency* with *field workability,* as it applies to general concreting field operations, is worthy of serious consideration.

The water-cement ratio theory of manufacturing concrete will continue to spread and with its more general adoption will come the natural striving for the most economic mixtures to obtain maximum workability.

Contrary to the prevailing opinions of many, a sand usually considered as fine (passing a No. 8 sieve and retained on a No. 100 sieve with the proper practical grading of the in-between sizes), *when used in the proper proportions,* with properly-graded coarse aggregates, will produce the maximum field workability and a more impermeable and therefore more durable concrete than can be produced with the use of sands of coarser gradings.

The grading of the particles of the sand are much more important than is generally conceded and the ultimate advance in the art of manufacturing concrete of the best quality requires a change from the present practice of mortar tests of sands on the basis of consistencies requiring various percentages of water instead of a mortar test with a fixed water-cement ratio and with variable proportions of the sand according to the sizes of the particles of the sand to be tested.

Mr. Wig. R. J. WIG.—Several years ago I had occasion to examine a great many concrete structures along the Pacific coast. The wonder to me has been not only then but during later years that cement will withstand all the abuse it gets. I would like to agree with the conclusion of the committee with respect to fine sand. In general I think the committee's conclusions are correct, that the sand should be coarse and graded, but we do have instances wherein fine sand has been used in sea-water work and

the structures have been durable. There is one case I recall in San Diego, the Spreckles wharf, in which they used a 1 : 2 : 3 or 1 : 2 : 4 concrete to form a jacket on the wood piles. This jacket was 5 or 6 in. thick, made from beach sand, and when I last examined it had been there 21 years, and was still standing. This would seem to indicate that it is not porosity alone that determines the durability of concrete in sea water.

MEASUREMENT OF AND ESTIMATING CONCRETE.*

Submitted by Committee C-5.

The report of Committee C-5, submitting a Tentative Standard covering "Measurement of and Estimating Concrete" was approved by the committee, and presented at the convention of the Institute in 1925.

Since that time there have been no changes in the Standard as submitted, and the final vote of the committee stood 15 for final adoption of the report as a standard, with only two dissenting votes.

In view of the above, the committee recommends that the tentative standard be submitted to a vote of the membership of the Institute for adoption as a standard.

FRANK R. WALKER, *Chairman.*

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*Report adopted by convention Feb. 24, 1926. Accepted as standard by letter ballot canvassed May 27, 1926.

AMERICAN CONCRETE INSTITUTE STANDARD.

STANDARD SPECIFICATION OF MEASUREMENT OF AND ESTIMATING CONCRETE—STANDARD METHODS FOR THE MEASUREMENT OF CONCRETE WORK.

As submitted by Committee C-5.

Serial Designation C-5A-26.

The following divisions are recognized as separate and distinct operations in the construction of concrete work for which separate modes of measurement are necessary.

I. *Plain and Reinforced Structural Concrete:*

(a) Concrete
(b) Forms
(c) Reinforcement
(d) Surface Finish
(e) Tile Fillers

II. *Sidewalks and Driveways.*

III. *Structural Precast Concrete:*

(a) Concrete
(b) Reinforcement
(e) Erection

IV. *Cast Ornamental Concrete Work.*

V. *Roads and Pavements:*

The following general rules shall govern the measurement of the above items (with the exceptions where specifically noted):

(a) All work shall be measured net as fixed or placed in the structure.
(b) In no case shall non-existent material be measured to cover extra labor.
(c) No allowance shall be made for waste, voids, or cutting.

I. PLAIN AND REINFORCED STRUCTURAL CONCRETE.

(a) Concrete.

1. The unit of measure for all concrete shall be the cubic yard.

2. All concrete shall be measured net as placed or poured in the structure, except as mentioned under Rule No. 4.

3. In no case shall an excess measurement of concrete be taken to cover the cost of forms or extra labor in placing.

4. All openings and voids in concrete shall be deducted with the following exceptions:

(a) No deduction shall be made for reinforcement, I-beams, bolts, etc., embedded in concrete except where a unit has a sectional area of more than 1 sq. ft.

(b) No deduction shall be made for pipes or holes in concrete having a sectional area of less than 1 sq. ft.

(c) No deduction shall be made for chamfered, beveled or splayed angles to columns, beams and other work, except where such chamfer, bevel or splay is more than 4 in. wide measured across the diagonal surface.

5. Each class of concrete having a different proportion of cement, sand or aggregate shall be measured and described separately.

6. Concrete in the different members of a structure shall be measured and described separately according to the accessibility, location or purpose of the work.

7. Concrete with large stones and rocks embedded in same (cyclopean masonry) shall be measured and described according to the richness of the mix and the percentage of rock in same.

8. Concrete stairs shall be measured by the linear foot of riser. Price to include forms, concrete, steel and finish tread. (Rise and tread to be given.) Finished surfaces shall be measured separately. In addition to stairs, measure platforms in square feet. Where stair risers are more than 5 ft. in length or stairs have unusual features, the concrete, forms and reinforcing should be measured and priced separately.

(b) Forms.

9. The unit of measure for formwork shall be the square foot of actual area of the surface of the concrete in contact with the forms or false work.

10. Forms to different parts of a structure shall be measured and described separately according to the position in the structure, accessibility, purpose and character of the work involved.

11. Forms shall in every case be measured and described separately and in no case (except as Rule 8) shall the measurement of concrete include the forms.

12. No deduction shall be made in measurement of surface of concrete supported by forms, because of forms being taken down and re-used two or three times in the course of construction.

13. The unit price for superficial measurement of forms shall include the cost of struts, posts, bracing, bolts, wire, ties, oiling, cleaning and repairing forms. No measurement to be made of these. Story heights over 12 ft. shall be listed separately, stating story height and area of forms.

14. Distinction shall be made between wood and metal forms.

15. Angle fillets or bevels to beams and columns shall be measured and described separately.

16. No deduction in measurement of forms shall be made for openings having an area of less than 25 sq. ft.

17. No deduction shall be made in floor forms for heads of columns of any shape, or area of clay or metal tile.

18. No deduction shall be made in column and girder forms for ends of girders, cross beams, etc.

19. No allowance shall be made for hand-holes in column forms for clearing out rubbish.

20. The measurement of column forms shall be the girth multiplied by the height from the floor surface to the under side of floor slab above or to the bottom of the drop panel.

21. Forms to rectangular, octagonal, hexagonal and circular columns shall be measured and described separately. Circular columns shall be listed separately, stating number, size and height of each.

22. Caps and bases to columns and other ornamental work shall be measured by number and fully described, giving overall dimensions.

23. The measurement of beam forms shall be the net length between columns multiplied by the sum of the breadth and twice the depth below the slab, except for beams at edge of floor or around openings which shall have the thickness of floor added to the sum of the breadth and twice the depth. The form area for the breadth of the beam should be deducted from the floor forms.

24. Allowance shall be made by number for pockets left for future beams.

25. Forms to circular work shall always be measured separately from forms to straight work.

26. No measurement or allowance shall be made for construction joints in slabs or beams, to stop the day's concreting.

27. Construction joints or expansion joints to dams, bridges and other large masses of concrete shall be measured by the square foot as they occur.

28. Falsework and staging for bridges, domes and other special work shall be described and measured separately.

29. Forms to cornices and moldings shall be measured by the lineal foot and the girth stated. (The term girth shall be taken to mean the total width of all curved and straight form surfaces touched by the concrete.) Plain forms to back of cornice to be measured separately.

30. Forms to window sills, copings and similar work shall be measured by the lineal foot. Indicate the dimensions.

31. If forms are required for the upper side of sloping slabs, such as saw-tooth roofs, they shall be measured separately.

(c) Reinforcement

32. The unit of measure of reinforcement shall be the weight in tons.

33. The weight shall be calculated on the basis of a square rod 1 in. x 1 in. x 12 in., weighing 3.4 lb.

34. Steel rods for reinforcement shall be measured as the gross weight required to be purchased.

35. Deformed bars shall be measured separately from plain.

36. Spirals shall be listed separately.

37. Separation shall be made according to accessibility, location or purpose of reinforcement.

38. The rods of each different size shall be measured and described separately.

39. Bent bars shall be measured separately from straight bars.

40. Chairs, ties, pipe sleeves, turnbuckles, clamps, threaded ends, nuts and other forms of mechanical bond shall be measured separately by number and size and allowed for in addition.

41. Wire cloth, expanded metal and other steel fabrics sold in sheets or rolls shall be measured and described by the square foot. The size of mesh and weight per square foot of steel in tension shall be stated. Allowance shall be made for waste, cutting, laps, etc., stating allowance.

(d) Surface Finish.

42. The unit of measure for finish or treatment of concrete surfaces shall be the square foot. The following shall be measured and described separately.

Cement wash. (State how many coats.)
Rubbing with carborundum.
Patching and removing of fins.
Scrubbing with wire brushes.
Tooling.
Picking.

Plastering.
Treating with acid.
Floor hardener.

43. Allowance shall be made for going over concrete work after removal of forms and patching up voids and stone pockets, and removing fins. Where rubbing is specified the cost of removing fins should be included with rubbing.

44. Cement or granolithic finish shall include all labor and materials for the thickness specified. Do not make deductions for partitions or columns. Do not deduct openings containing less than 25 sq. ft.

45. Finish laid integral with the slab shall be measured separately from finish laid after the slab has set.

46. Allowance shall be made for protection of finish with sawdust, sand or tenting, giving area to be covered.

47. Grooved surfaces, gutters, sills, curbing, etc., shall be measured separately from plain work and shall be measured by the square foot or lineal foot as the case may require.

48. Cement cove and base shall be measured by the lineal foot, giving size.

(e) Tile Fillers.

49. (a) Clay tile should be estimated by the number of square feet of each size tile required. Give number of end tile.

(b) Gypsum tile shall be measured by the number of lineal feet of each size. Soffit pieces shall be measured by the lineal foot, stating width.

(c) Metal tile shall be measured by the number of lineal feet of each size. State number of end tile pieces necessary.

(d) Wood tile shall be measured by the number of lineal feet of various depths.

(e) Metal lath or other ceilings under wood or metal tile ceilings shall be measured in square feet of surface covered.

II. SIDEWALKS AND DRIVEWAYS.

50. Sidewalks and driveways shall be measured by the square foot; the thickness to be stated in the description.

51. All linear surface measurement shall be made along the grade line or the actual length of the sidewalk or driveway and not merely horizontally.

52. Finish, reinforcing and lining in squares and cinder or stone foundations shall not be separately measured, but shall be described. Expansion joint material to be measured separately per linear foot.

53. Curbs and curb and gutter work shall be measured by the lineal foot and separated according to character and size, and shall include foundations, forms and finish.

54. In measuring curbs the full height and width or thickness of same shall be taken to the extreme edge.

55. Circular corners to curbs and gutters shall be measured separately by number, stating radius and length measured on the curve.

56. Vault lights shall be measured by the square foot, the size and type of glass, forms, steel and finish to be described but not measured separately. Beams under vault lights shall be measured by the lineal foot. In measuring vault lights the measurement shall be taken at least 4 in. beyond the outside line of the glass in each direction.

III. STRUCTURAL PRECAST CONCRETE.

(a) Concrete.

57. The term structural precast concrete is taken to include unit construction by the various systems.

58. The unit of measurement for structural precast concrete shall be the cubic foot, and shall be measured net as provided for monolithic concrete.

59. The various members shall be measured on the ground before erection.

60. No measurement shall be taken of forms.

(b) Reinforcement.

61. Reinforcement shall be measured separately as provided in Paragraphs 35 to 47, inc.

(c) Erection.

62. The unit of measure for the erection of structural precast concrete shall be the volume of the finished member in cubic feet.

63. The unit of measurement for grouting shall be the cubic foot of grout required.

IV. CAST ORNAMENTAL CONCRETE WORK.

64. Cast concrete shall be measured by the cubic foot, but the measurement shall be the smallest rectangular solid that will contain the piece measured and not its actual content.

65. No allowance shall be made for forms.

66. No allowance shall be made for reinforcement in trim and ornamental work.

67. No allowance shall be made for surface finish in trim and ornamental work.

68. Circular work shall be measured separately from other work.

69. Mitre blocks and end blocks for cornices, etc., shall be measured separately from straight molded work.

70. Vases, seats, pedestals, balusters and other similar items shall be measured by number and description with overall dimensions.

71. The unit of measure for the erection of ornamental concrete shall be the volume in cubic feet as measured under Rule 64.

V. ROADS AND PAVEMENTS.

72. The unit of measure for concrete pavement on roads or streets shall be the square yard of pavement surface.

73. All linear surface measurements shall be made along the grade line or the actual length of the pavement, and not merely horizontally.

74. Deductions shall be made for openings in concrete pavement of more than one square yard surface area. No deductions shall be made for individual openings, such as manholes, catch basins, inlets, lampholes, monument covers, etc., of less than one square yard surface area.

75. Concrete roads and pavements having sections of varying proportions of cement, sand and stone, or having sections of varying thickness or cross-section, shall be measured and described separately.

76. Concrete roads and pavements containing reinforcement of any type shall be measured and described separately from that not containing reinforcement.

77. Concrete roads and pavements requiring forms shall be measured and described separately.

78. Concrete roads or pavements requiring any special surface treatment shall be measured and described separately.

Report of Committee P-4, Concrete Staves.

At the 1924 meeting, the American Concrete Institute adopted as tentative "Specifications and Building Regulations for Concrete Staves" submitted by this committee. These with supplementary notes were published in the 1924 *Proceedings*.

No report was presented by this committee at the 1925 meeting. At that time an extension of time was requested to complete additional tests to determine fully the practicability of the strength and absorption requirements as given in the tentative specifications. Such tests were made in the plants of the Waterloo Concrete Corp., Waterloo, Iowa; the Michigan Silo Co., Peoria, Illinois; the Interlocking Cement Stave Silo Co., Wichita, Kansas; the Neff and Fry Co., Camden, Ohio; and in several other plants. A description of the tests at Wichita which are representative is included as a part of this report.

On the basis of results obtained from these tests your committee respectfully recommends that the present tentative specifications be adopted as standard.

In support of the specifications we are submitting the following data covering a series of tests made on staves in the plant of the Interlocking Cement Stave Silo Co., Wichita, Kansas. These were conducted under the direction of A. A. Anderson, construction engineer, attached to the Kansas City office of the Portland Cement Association.

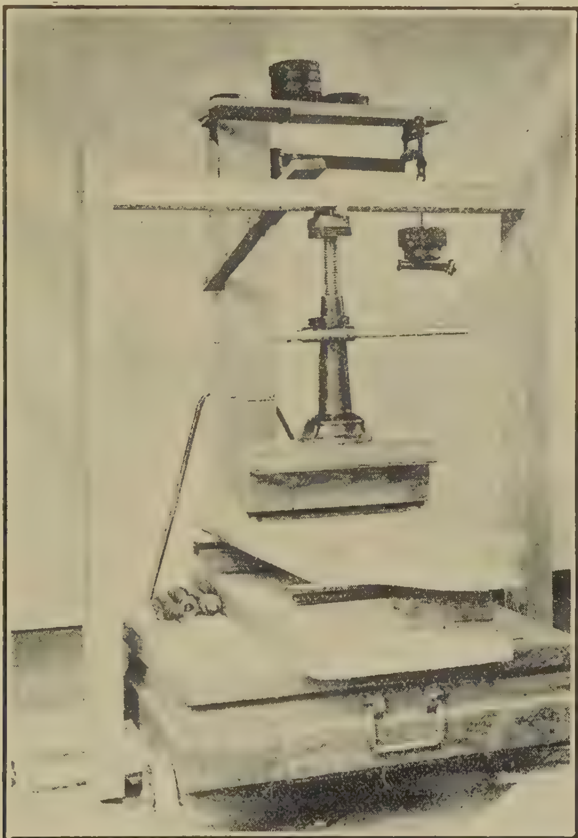
The transverse tests were made in a specially devised machine shown in Fig. 1. The staves were supported on one end by one ⅝-in. round rod and at the other end by two ⅝-in. round rods set at right angles to each other providing a spherical bearing for equalization of the loading. The distance between supports was 22 in. The load was applied at mid span with a screw jack through a spherical bearing obtained by two ⅝-in. round rods placed at right angles to each other. Metal plates ½ in. x 1½ in. shimmed with blotting paper were placed between the stave and the supports and between the stave and the point where load was applied at mid span. All specimens were tested 28 days after being manufactured.

Absorption tests were made on sections cut from staves, with all four edges being broken edges. These sections had a face area of approximately 90 sq. in. The specimens were dried to constant weight in an oven and then immersed in water at room temperature (70 to 80 deg. F.) for 48 hours. They were then dried with a towel of all surplus water and weighed. The increase in weight, expressed as a percentage of the dry weight gave the absorption.

Specimens were made under ordinary plant conditions. The staves were of the "Interlocking type" 28 in. long, 10 in. wide and 2½ in. thick. Owing to the ends being of the O and G type it was possible to secure only

a 22-in. span in the testing machine. Results were then translated to equivalent 24-in. span as required in the specifications.

In Series A the aggregate consisted of 80 per cent sand and 20 per cent chat. Table I gives information as to mixture, fineness modulus of aggregate, yield per sack of cement and the results of the transverse and



STAVE-TESTING MACHINE USED IN TESTS OF THE INTERLOCKING CEMENT STAVE SILO CO., WICHITA, KANSAS.

absorption tests. These results are further summarized in Table II. It is to be noted that the staves in Series A did not meet transverse requirements—Absorption, however, was below the maximum of 6 per cent allowed.

In Series B the percentage of chat was increased to 30 per cent by weight producing a fineness modulus of the mixed aggregate of 3.9. This

TABLE I.—TESTS OF CONCRETE STAVES, INTERLOCKING CEMENT STAVE SILO COMPANY, WICHITA, KANSAS.

Dimensions of Staves—28 x 10 x $2\frac{1}{2}$ in. with O and G Ends.
All Staves Made with Anchor Automatic Tamper Batch Mixer.

Series	Specimen.	Mixture		Aggregate Composition, Per cent by wt.	F. M.	Number per Bag.	Time of Mix, min.	Method of Curing.	Transverse Strength			Absorption, per cent
		By Wt.	By Vol.						22-in. Span.	Per inch Width.	Equiv. 24-in. Span.	
A....	1	1:5	1:4.2	20 Chat 80 Sand	3.7 3.7	12	5	Sprinkled 27 days	700	70.0	64.2	
	2	"	"	" "	"	"	"	" "	915	91.5	83.9	3.5
	3	"	"	" "	"	"	"	" "	810	81.0	74.2	
	4	"	"	" "	"	"	"	" "	910	91.0	83.5	
	5	"	"	" "	"	"	"	" "	850	85.0	77.9	
B....	1	1:5	1:4.2	30 Chat 70 Sand	3.9	10	5	" "	1150	115.0	105.5	
	2	"	"	" "	"	"	"	" "	1100	110.0	100.8	4.0
	3	"	"	" "	"	"	"	" "	1110	111.0	101.8	
	4	"	"	" "	"	"	"	" "	1200	120.0	110.0	
C....	1	1:4	1:3.3	20 Chat 80 Sand	3.7	8	5	" "	1280	128.0	117.3	
	2	"	"	" "	"	"	"	" "	1300	130.0	119.1	
	3	"	"	" "	"	"	"	" "	1220	122.0	111.8	2.9
	4	"	"	" "	"	"	"	" "	1125	112.5	103.1	
D...	1	1:5	1:4.2	30 Peb. 70 Sand	4.0	10	5	" "	1000	100.0	91.7	4.8
	2	"	"	" "	"	"	"	" "	950	95.0	87.1	
E....	1	1:4	1:3.2	Native R. Sand	3.2	8.5	4	" "	900	90.0	82.5	
	2	"	"	"	"	"	"	" "	900	90.0	82.5	5.1
F....	1	1:5	1:4	"	3.2	9.5	4	" "	835	83.5	76.5	
	2	"	"	"	"	"	"	" "	860	86.0	78.8	3.0
G....	1	1:7	1:5.6	"	3.2	15.0	4	" "	450	45.0	41.3	
	2	"	"	"	"	"	"	" "	490	49.0	45.0	6.7
H...	1	1:3.6	1:3	Little R. Sand A. V. I. Pit	3.7	7.5	By hand	" "	1100	110.0	101.0	3.1
	2	"	"	"	"	"	"	" "	1070	107.0	98.0	
I....	1	1:4.8	1:4	"	3.7	10.0	"	" "	1000	100.0	91.7	
	2	"	"	"	"	"	"	" "	1005	100.5	92.1	3.9
J....	1	1:5.9	1:5	"	3.7	12.5	"	" "	885	88.5	81.0	
	2	"	"	"	"	"	"	" "	1020	102.0	93.5	4.5
K...	1	1:8.3	1:7	"	3.7	17.0	"	" "	650	65.0	59.7	
	2	"	"	"	"	"	"	" "	670	67.0	61.4	5.6
L....	1	1:3.7	1:3	L. R. S. 10 Blow Sand	3.5	7.5	"	" "	905	90.5	83.0	
	2	"	"	"	"	"	"	" "	870	87.0	79.7	4.3
M...	1	1:4.9	1:4	"	3.5	10.0	"	" "	705	70.5	64.5	
	2	"	"	"	"	"	"	" "	1015	101.5	93.0	3.0
N...	1	1:6	1:5	"	3.5	12.5	"	" "	700	70.0	64.2	
	2	"	"	"	"	"	"	" "	750	75.0	68.7	5.3

aggregate was beyond the limit of coarseness for satisfactory workability and produced a stave too rough to be salable. The yield, strength and absorption were satisfactory.

Aggregate in Series C was composed of 20 per cent chat and 80 per cent by weight of local sand. The mixture in this series was 1 : 4 by weight or 1 : 3.3 by volume, dry and rodded. This series gave the highest transverse strength and the lowest absorption of the entire test. The yield was 8 staves per sack of cement.

Owing to the harsh working concrete resulting from inclusion of chat in the aggregate 30 per cent of pebbles was added to the local sand in Series D. This produced an aggregate of fineness modulus of 4.0. This concrete worked satisfactorily in the machine producing a stave of satisfactory appearance but fell slightly under the transverse strength requirements.

TABLE II.—SUMMARY OF TESTS, SILO STAVES, INTERLOCKING STAVE SILO COMPANY, WICHITA, KANSAS.

Series.	Mix by Volume.	F. M.	Number of Staves per Bag.	Method of Curing.	Time of Mix, minutes.	Transverse Test, 24-in. Span.	Absorption, per cent.
A	1.42	3.7	12	Sprinkled 27 days	5	76.7	3.5
B	1:4.2	3.9	10	"	5	104.5	4.0
C	1:3.3	3.7	8	"	5	112.8	2.9
D	1:4.2	4.0	10	"	5	89.2	4.3
E	1:3.2	3.2	8.5	"	4	82.5	5.1
F	1:4	...	9.5	"	4	77.7	3.0
G	1:5.6	...	15.0	"	4	43.2	6.7
H	1:3	3.7	7.5	"	By hand	99.5	3.1
I	1:4	3.7	10.0	"	"	91.9	3.9
J	1:5	3.7	12.5	"	"	87.2	4.5
K	1:7	3.7	17.0	"	"	60.5	5.6
L	1:3	3.5	7.5	"	"	81.3	4.3
M	1:4	3.5	10	"	"	78.7	3.0
N	1:5	3.5	12.5	"	"	66.4	5.3

Series E, F and G were made up of local river sand which is available in abundance. The average of this series fell under the transverse strength requirements, with Series G having an excess of absorption also.

In Series H to K inclusive pit-run aggregate from the Little Arkansas River was used. This aggregate was well graded from coarse to fine except for a scarcity of material below a 50 sieve. This aggregate however worked satisfactorily in the machine producing a stave somewhat granular in appearance but not objectionable. The staves in Series H and I passed all requirements of the specifications.

In Series L to N inclusive 10 per cent of blow sand was added to the pit-run material from the Little Arkansas River to relieve the somewhat harsh workability of the concrete. No particular advantage was apparent with the finer sand but the transverse strength and amounts of absorption were adversely affected.

W. O. BRASSERT, *Chairman.*

W. G. KAISER, *Secretary.*

AMERICAN CONCRETE INSTITUTE STANDARD.

Standard Specifications and Building Regulations for Concrete Staves.*

Submitted by Committee P-4 on Concrete Staves.

(Serial Designation P-4A-26)

General. 1. Concrete staves meeting the requirements of the following specifications may be used in the construction of silos, coal pockets, corn cribs, grain bins and other structures for which these units are suitable.

Tests. 2. Concrete staves must be subjected to transverse and absorption tests. All official tests must be made in a testing laboratory of recognized standing. Six samples representing the ordinary commercial product selected at random from stock must be provided for the purpose of testing.

Transverse Strength. 3. The ultimate transverse strength of the test staves at 28 days after being manufactured or when shipped must average not less than 90 lb. with no test falling below 75 lb. for each inch of width of the stave.

4. The absorption at 28 days after being manufactured or when shipped must not in any case exceed 6 per cent.

Absorption. 5. The transverse test shall be made as follows: The sample to be tested shall be placed flatwise in the testing machine and supported at one end on a ⅜-in. round rod and at the other end by a spherical bearing block using a steel plate 2 in. wide and of sufficient stiffness to properly distribute the load between concrete and load point. The distance between points of support shall be exactly 24 in. The load shall be applied at mid-span through a spherical bearing block.

Transverse Test. 6. In the absorption tests the samples shall be first thoroughly dried to a constant weight at a temperature not to exceed 230 deg. F. After drying, the sample shall be completely submerged in clean water at a temperature of between 60 and 80 deg. F. for a period of 48 hours. The specimen shall then be removed, the surface water wiped off, and the sample reweighed. The percentage absorption is the weight of the water absorbed divided by the weight of the dry specimen and the quotient multiplied by 100.

Vertical Loading. 9. The load on any concrete stave wall, including the superimposed weight of the wall, shall not exceed 200 lb. per sq. in.

Lateral Loading. 10. Silos, grain tanks, coal pockets, corn cribs, etc., constructed of concrete staves shall be hooped with steel rods or bands of such cross-sectional area that the steel will not be stressed to exceed 16,000 lb. per sq. in., and the hoops shall be placed at such intervals that the staves will not be loaded to exceed 25 per cent of their average transverse strength.

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*Adopted by convention Feb. 25, 1926, to be referred to letter ballot for acceptance as standard. Letter ballot canvassed May 27, 1926.

Report of Committee P-1, Standard Building Units.

The practice of having their products tested periodically is becoming very general among the producers of concrete masonry units. Since in a few cases, products manufacturers have reported that uniform results were obtained in one laboratory while variable results were obtained from another and since occasionally reports have been received where cardboard, wood or similar materials were used for capping purposes, the committee decided to give further study to that portion of our Standard Specifications for Concrete Building Block, Building Tile and Brick which relates to the testing of these units for compressive strength.

In this connection it was found desirable to propose the addition of more explicit instructions in three paragraphs. The changes will be of special value to the laboratories which have had little experience in testing these products.

The revised sections are hereby submitted as tentative amendments to the corresponding sections of the present specifications.

Proposed amendment to Paragraph 16, Section 2 of the Standard Specifications for Concrete Building Block and Concrete Building Tile, (Serial Designation *P-1A-25*) also to Paragraph 12, Section 2 of the Standard Specifications for Concrete Brick, (Serial Designation *P-1B-25*).

"Bearing surfaces shall be made plane by capping with plaster of paris or a mixture of one-half portland cement and one-half plaster of paris, which shall be allowed to thoroughly harden (from 3 to 6 hours) before the test. No point on the surface shall deviate from the plane more than 0.003 in. The cap shall not be thicker than ⅛ in. It shall be formed by means of an accurately machined metal plate or a heavy plate glass having a true surface."

Proposed amendment to Paragraph 18, Section 2 of the Standard Specifications for Concrete Building Block and Concrete Building Tile, (Serial Designation *P-1A-25*), also to Paragraph 14, Section 2 of the Standard Specifications for Concrete Brick (Serial Designation *P-1B-25*).

"The load shall be applied through a spherical bearing block placed on top of the specimen. The rate of loading after 50 per cent of the ultimate load has been applied shall not be greater than that which will produce a shortening of the specimen of .02 inch per minute."

Proposed amendment to Paragraph 20, Section 2 of the Standard Specifications for Concrete Building Block and Concrete Building Tile (Serial Designation *P-1A-25*).

"Machined-steel or cast-iron plates of sufficient thickness to prevent appreciable bending shall be placed between the spherical bearing block and the specimen. In no case shall the distance between the edge of the spherical bearing block and the end of the bearing plate be greater than twice the thickness of the plate. Where a number of thin plates are used, in no case shall the plates be less than one inch thick nor shall any plate extend

beyond the one immediately above it a greater distance than twice the thickness of the plate."

Discussion.—The new provisions define more closely three factors which have an important bearing upon the ultimate strength of a test specimen; namely, the thickness and trueness of the caps, the rate of applying the load and more definite requirements for the plates used between the spherical bearing block and the specimen.

The thickness of the caps and the trueness of the bearing surface are especially important factors. Tests carried out at the Structural Materials Research Laboratory (See Bulletin 14, "Effect of End Condition of Cylinder on Compressive Strength of Concrete" by H. F. Gonnerman) on 6 x 12-in. concrete cylinders made on and capped with sheared plates 0.004 to 0.012 in. out of true showed reductions in strength as high as 32 per cent. Attention of the committee has been called frequently to the low results obtained from test specimens on which exceptionally heavy caps were used. Often the capping material is considerably weaker than the product being tested and, where thick caps were used, failure or flow of the caps occurs which results in an uneven distribution of the load.

A considerable variation is found in testing concrete specimens for compression when the rate of loading is varied, more especially after more than half of the load has been applied. Prof. D. A. Abrams in an article on "Rate of Application of Load on Compressive Strength of Concrete" *Proceedings*, A. S. T. M. Vol. XVII, Part II, 1917, states that "A machine speed which gives a shortening of the test piece of 0.01 to 0.02 in. per minute is recommended as a standard for compression tests of concrete." Upon this basis the committee has considered it desirable to specify a maximum rate of loading.

Occasionally plates as thin as one inch have been used between the spherical bearing block and the specimen. Calculations of the deflection of the plates indicate that a bearing plate less than 2½ in. thick may deflect sufficiently to transmit relatively high loads directly under the bearing block and relatively low loads at the ends of the specimen. Often it is convenient to use more than one plate and the proposed specification for these plates will safely permit this method.

Last year, tentative standard amendments to the Standard Specifications for Concrete Building Block, Building Tile and Brick were adopted which clarified the wording of some sections. This year, your committee submits the amendments found in this report for tentative standard amendments. Next year we will request that all of these tentative amendments be adopted as standard. Exhibits "A" and "B," accompanying this report are the present Standard Specifications into which all the proposed amendments have been written.*

C. L. BOURNE, *Secretary.*

*Specifications, adopted as standard in 1925, are published as a separate print. Tentative amendments presented at 1926 convention do not disturb existing standards.

Report of Committee T-1, on Crazing.

As far as the report of the Committee on Crazing is concerned I am almost in the position of the usual chairman in reporting progress, but I want to give you some little idea of what we have done. In the first place we did try to do some real work, but it only concerns what we predicted last year. We got some data which form simply a post-mortem, and like post-mortems, it is useless because we cannot go back and get the immediate data which will enable us to form some idea of why what happened did happen at all.

It is all very well to be told by a contractor that he used a certain mix or a certain amount of water. According to those statements he should or should not have had crazing, and he invariably got the opposite, so those types of examination are not interesting and must and will continue to be made. So far as a basis for getting recommendations, they are out of the question.

For about six weeks we have had a man working practically full time on laboratory work on crazing. We will continue this, as far as present indications go, indefinitely. We have also the promise of assistance from another laboratory which will devote considerable time to this work, and also we have the promise of assistance of another group in making some field tests of patterns based upon laboratory work. We intended to follow this thing along, but, as we cautioned you before, it is a long, long drawn out problem which cannot be brought to any definite conclusion simply by the opinions of any of us or any opinions offered to us. It is a laboratory problem and conclusions must be based upon laboratory data, corroborated by further field tests.

P. H. Bates, *Chairman.*

DISCUSSION.

Mr. Hatt PROF. W. K. HATT.—I have been interested to find out just what was a craze and what was an actual separation or fissure in a concrete surface. We have made some observation on crazed beams of various types at the laboratory, to determine the development of surface fissures due to various degrees of extension of the surface. The beams had been exposed to the weather for six months after three days' curing under wet burlap. There are three intensities of surface crazing patterns formed without the coming out of laitance, the badly crazed, the moderately crazed and free from crazing. There are two methods of troweling.

These observations will be carried on still further to determine the relation between these fine patterns you so often see on the surface of sidewalks and the subsequent behavior of the surface. (See Prof. Hatt's paper "Extensibility of Concrete," p. 364.)

Cast Stone and Architectural Concrete.

Submitted by Committee P-2.

Committee P-2 has to do with concrete products. Just what constitutes a concrete product seems by common consent to be clear enough to the members of this Institute, but I do not think a distinction between concrete products and other kinds of concrete has been officially made. A definition will at some time be necessary and will be made by the Institute. But until the Institute formally defines a concrete product I will continue to think of it as a thing made with concrete, in the making of which workmanship is more important than material. In other things made with concrete the material always seems to be more important than the workmanship.

I do not wish you for a moment to think that I am suggesting this distinction as the basis of a definition for concrete products, because I, myself, am little concerned with definitions and do not think that this would be a good basis for one. But I may use this distinction as a device to direct your thoughts towards a fact, namely; that the Institute in the past has given little or no attention to craftsmanship. Much consideration has been given to materials but little to the control of materials. Considerable amounts of information have been recorded on the design of concrete structures, on the behavior of these structures in their daily work, but little on the technique or craftsmanship by which they were built.

This committee, then, is a committee on craftsmanship. It is a new committee and is in a large measure a kind of free lance among the committees of the Institute. Its work will differ greatly from what is usual. It may never write a specification nor a recommended practice. It may not present standards for publication, nor even report every year. But it will try to bring before the makers of concrete things the value of good workmanship, of fine technique as exemplified by good work, well done.

The committee intends—

(1) To write into the *Proceedings* of the Institute a record of the present state of the art of making architectural concrete and to supplement that by a record of developments as they occur.

(2) To identify certain scientific knowledge now in the Institute with certain known processes in the art. To indicate certain well-known processes in the art which have not been and should be translated into scientific forms.

(3) To record the opinion of architects concerning concrete as an architectural medium in order that the changes in their opinion from what is now generally unfavorable to what will in the future be favorable may serve as mile posts by which to measure our progress.

Some of this work the chairman is equipped to do. Some of it must be done by others whose knowledge of a particular technique, or method of handling material for a particular purpose, is greater than his. I do not know how large the committee will be. For the present it will consist of the chairman and his conscience, who are expected to attend all meetings and to work in harmony. As the work begins to take orderly form craftsmen will be needed to tell how their work is done, artists will be needed to analyze and record the knowledge of the craftsmen as well as to give to them in turn new knowledge and better materials. And architects will be needed to give the benefits of all this to the building industry.

The work of this committee will not be as general nor as far reaching as it could be, because it will necessarily be limited by the limitations of the chairman, who has been trained in the execution of architectural design almost to the exclusion of everything else. Therefore, the committee's attention will be confined to those concrete products, which belong to the group related to architectural concrete, such as concrete surface treatments and precast stone. Furthermore, the committee will begin its work in the present concerning itself with the processes of today, and it will gradually as time and opportunity allow include such information as may be gleaned from the past proceedings of the Institute and from analogy between the technique of concrete and that of other plastic materials.

The work will be presented to the Institute in the form of a committee report, or a paper, or a suggestion for papers on a certain subject by those who are fitted to present them.

On the whole, I think the work will be interesting, and if the hopes of the chairman are realized, to a degree which is not impossible, it will be useful as well.

JOHN J. EARLEY, *Chairman.*

Concrete Products Plant Operation.

As Reported by Committee P-6.

Committee P-6 felt that it would be desirable to elaborate on the curing tests reported at the 1925 convention to cover curing conditions during the whole year. A sub-committee on curing was appointed which outlined a séries of tests to determine the effect of weather conditions on the outdoor curing of concrete block.

Monthly tests were started in November, 1925 and will be carried through October, 1926.

The attached table shows the outline of tests as adopted by the committee.

Up to the present time 120 block have been made and 48 have been tested.

The test program is under the direction of Earle D. McKay, of Universal Portland Cement Co.

The block were manufactured, cured and stored at the plant of the Diamond Block Co., St. Paul, Minn. They are being tested at the Experimental Engineering Building, University of Minnesota, under the supervision of Professor M. B. Lagaard.

Benjamin Wilk, *Chairman.*
C. L. Bourne, *Secretary.*

OUTLINE OF PROPOSED TESTS ON CURING OF CONCRETE BLOCK.

Submitted by Committee P-6.

Compression tests of 8 x 8 x 16-in. 3-core concrete building block.
Mix 1:7 by volume of dry and rodded combined aggregates; aggregates measured separately.
Cement: portland.
Aggregate: a mixture of sand (0 to No. 4) and pebbles (No. 4 to ½ in.) (Fineness Modulus about 4.25).
Consistency as wet as practicable. (Mixing water to be accurately measured.)
Machine mixed concrete (½ bag batch).
Concrete to be mixed 3 min. after all materials, including water, are in mixer.
Block tamped the same number of blows and made by same operator if possible.
Four blocks are to be selected for test for each condition from at least 3 batches.
Block cured as indicated.
Blocks while outdoors to be protected from sun by additional blocks.

Group	Month	Approximate Outdoor Temperature deg. F.	Curing Condition	Age at Test	Number of Specimens	
					Made	Tested
1	June July* August September	Above 65	(*a*) In moist room for 24 hours, then out-doors and sprinkled once daily for 7 days. (*b*) In steam room at 100° to 125° F. for 24 hours, then outdoors and sprinkled once daily for 7 days.	28 days 3 mo.	150	80
2	October† April May	Below 65 Above 45	(*c*) In moist room at about 70° F. for 24 hours, then outdoors and sprinkled once daily for 7 days. (*d*) In unheated moist room for 24 hours, then outdoors and sprinkled once daily for 7 days.	28 days 3 mo.	120	64
	November December‡ January February March	Below 45	(*e*) In moist room at about 70° F. for 24 hours, then outdoors, but not sprinkled. (*f*) Store in steam room at 100° to 125° F. for 24 hours, then outdoors, but not sprinkled	28 days 3 mo.	180	96
					450	240

* Repeat (*a*) and (*b*) during July but store in Laboratory after sprinkling for 7 days outdoors.
† Repeat (*c*) and (*d*) during October, but store in Laboratory after sprinkling for 7 days outdoors.
‡ Repeat (*e*) and (*f*) during January but store in Laboratory at about 70° F. upon removal from curing room.

October 14, 1925.

Report of Committee S-1 on Reinforced-Concrete Chimneys.

The reorganization of Committee S-1 on Reinforced-Concrete Chimneys was approved by the Board of Direction at its May, 1925, meeting, but due to some misunderstanding as to the organization of the committee it was not released for active work until June. The committee assignment was the preparation of a standard specification for the design and construction of reinforced-concrete chimneys and the conducting and studying of tests of the structural behavior of such chimneys in operation.

The committee was divided into two subcommittees, one on Standard Specifications, composed of E. A. Dockstader, chairman, F. P. Fairchild, William Jassoy, Prof. E. R. Maurer, and I. F. Stern; and one on Tests and Research, composed of J. W. Lowell, chairman, E. A. Dockstader, A. C. Irwin and Prof. E. R. Maurer.

Due to a change in business connection during the year J. W. Lowell was obliged to resign as chairman of the subcommittee on Tests and Research, since which time this work has been carried on under the acting chairmanship of Benjamin Wilk, assistant western manager, Service Bureau, Universal Portland Cement Co. Prof. M. B. Lagaard of the University of Minnesota has co-operated actively with the committee and has been of especial help in connection with the strain gage tests of the Duluth chimney.

The subcommittee on Standard Specifications has prepared an outline for the proposed Standard Specification for the Design and Construction of Reinforced-Concrete Chimneys and is actively engaged in the preparation of this specification which has been divided into sections, each assigned to different members of the subcommittee. Progress to date indicates that a specification will be completed for submission to the Institute as a tentative standard before the next annual convention.

The subcommittee on Tests and Research has concentrated particularly on a very complete series of tests now under way on a new 300-ft. high reinforced-concrete chimney constructed by the Weber Chimney Co. for the Universal Portland Cement Co. at its Duluth, Minnesota, plant. The tests being made include:

1. An exhaustive series of tests of the temperatures within the concrete shell and lining at various locations in the height of the chimney, under varying conditions of operation.
2. Measurement of wind velocities adjacent to the chimney.
3. Measurement of wind pressures exerted on the chimney under different velocities.
4. Measurement of deflections of the chimney due to wind and heat.
5. Strain-gage measurement of stresses in the reinforcement and concrete of the chimney shell under varying temperature conditions and wind velocities.

The development of this series of tests has involved the design of new apparatus and equipment for accurate measurement and it is confidently hoped that the tests will furnish valuable information as to the actual behavior of chimneys under operation. These results should be of material assistance to the subcommittee on Standard Specifications in its recommendations for design, particularly in connection with stresses due to temperature changes which are the most important stresses requiring consideration in the design of a reinforced-concrete chimney. It is hoped that the results of the tests will be available in time to prevent delay in the preparation of this portion of the design specification.

A more detailed description of the tests of the chimney of the Universal Portland Cement Co. at Duluth is given in a paper on this subject presented at the 1926 convention of the Institute.*

The subcommittee on Tests and Research is also keeping in touch with other tests of chimneys being planned or being carried on in other locations and is endeavoring to interest owners and chimney constructors in the importance of such tests to furnish definite data on the behavior of chimneys in operation. Practically no reliable data, of this nature, are available and such data are essential to the preparation of intelligent and comprehensive recommendations for the design of such structures.

Great credit is due to the Universal Portland Cement Co. for conducting these tests on its Duluth chimney, which involve a great deal of time and care, as well as expense.

C. E. NICHOLS, *Chairman.*

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*See p. 350 for details of test.

Report of Committee E-3, on Research.

Introduction.—The reorganization of Committee E-3 on Research was authorized at a meeting of the Board of Direction held on April 28, 1924. The committee was organized late in 1924, the membership being made up so far as possible from universities and other institutions where research in the field of concrete is under way or may be undertaken. The functions of the committee are:

(1) To present to the Institute, from year to year, a summary of what the various educational institutions and other research agencies are doing, that may be related to the work of the Institute.

(2) To act as a means of contact with a view to persuading qualified institutions to undertake researches in concrete that would expedite the work of the various committees of the Institute.

In general, it is the duty of the committee to serve as a watchtower upon investigations of concrete with a view to reporting on researches needed and undertaken, and to give the Institute as complete information as possible on the work of research laboratories and educational institutions in the field of concrete.

As the committee was not organized until late in 1924, no report was presented at the 1925 convention. However, a meeting was held of the members in attendance at the 1925 convention and preliminary plans were made for the work of the year. The committee held no other meetings during the past year and its work has been carried on for the most part by correspondence.

Research Agencies and Status of Research Work in the United States.—There is considerable activity at present among the various agencies carrying out researches in the field of concrete and numerous important investigations are under way. The principal research agencies engaged in this work and the publications in which the results are published from time to time are as follows:

Research Agency	Results of Researches Published in
U. S. Bureau of Standards.	Technologic papers, circulars, letter circulars, papers by members of staff. For a complete list of publications on cement, concrete, reinforced concrete, stucco, building stone, and related subjects, see Letter Circular 155 of Feb. 15, 1925.

RESEARCH AGENCY	RESULTS OF RESEARCHES PUBLISHED IN
U. S. Bureau of Public Roads.	Monthly journal *Public Roads*,* papers by members of staff before engineering and technical societies, Association of State Highway Officials, and in technical journals.
State Highway Departments.	Monthly magazines by various highway departments, papers by members of staff before highway associations and engineering societies.
Universities and Colleges.	Bulletins, papers by members of staffs before engineering and technical societies, Summary and Index of Engineering Research Work at Land Grant Colleges published by the Engineering Section of the Land Grant College Association (1923).
National Research Council.	Bulletins, reprint and circular series, Proceedings of the Highway Research Board, bi-monthly magazine, "Highway Research News," Proceedings of the National Academy of Sciences.
Committees of National Engineering and Technical Societies.	Reports of committees in proceedings of the various societies.
Structural Materials Research Laboratory, Lewis Institute.	Bulletins and circulars of laboratory, papers before engineering and technical societies.
Navy Department, Bureau of Yards and Docks.	Public Works Bulletins of the Navy.

The U. S. Bureau of Standards has for many years been active in research on concrete and concrete materials and has published many papers on the results of this work. At the present time the Bureau is co-operating with the Portland Cement Association in an important and extensive investigation on the constitution of portland cement, further details of which are given in Part I of Appendix A of this report.

The U. S. Bureau of Public Roads for several years has been actively engaged in making researches on concrete in connection with federal-aid highway projects and has made important contributions to the literature on this subject. Various state highway departments, those of California, Illinois, Iowa, Michigan, Minnesota, and Pennsylvania in particular, have contributed much to our knowledge of the properties of concrete and are now carrying out important researches on a variety of subjects.

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*The November, 1925, issue of *Public Roads* gives a tabulation of highway research projects recently completed or now in progress under the auspices of state highway departments, universities and state engineering experiment stations.

Many universities and colleges, notably California, Columbia, Illinois, Iowa, Maryland, Michigan, Purdue, and Wisconsin, are also actively engaged in researches on plain and reinforced concrete and on concrete materials and have published numerous valuable reports. A summary and index of researches carried out at land grant colleges was published by the engineering section of the Land Grant College Association in 1923.

The National Research Council through its bulletins and circulars and through the Proceedings of the Highway Research Board has aided in the publication of timely researches on concrete, which publications contain valuable data.

There are a number of committees of national engineering and technical societies which are engaged in researches on cement and concrete. Committee C-1 on Cement and C-9 on Concrete and Concrete Aggregates of the American Society for Testing Materials have work under way which has an important bearing on test procedure and on specifications for various concrete materials. Committee C-1, at the last annual meeting of the society, presented a "Manual of Cement Testing" in the interests of greater uniformity in tests on portland cement. The Special Committee on Cement of the American Society of Civil Engineers is also taking an active interest in and is sponsoring investigations in this field. The Special Committee on Concrete and Reinforced-Concrete Arches of the American Society of Civil Engineers is carrying out extensive studies on arches under construction and elaborate tests of laboratory models. Committee S-1 on Reinforced-Concrete Chimneys of the Institute is making tests on a 309-ft. reinforced-concrete chimney and Committee P-6 on Concrete Products Plant Operation has under way a series of tests on the curing of concrete block. A committee of the Engineering Foundation is making extensive tests on a single-arch type dam of plain concrete near Fresno, Calif., under a great variety of conditions as to loads and temperatures.

Since its organization, Sept. 1, 1914, the Structural Materials Research Laboratory, at Lewis Institute, Chicago, has been active in concrete research and at present has under way a number of studies on plain concrete and related subjects.

Extensive tests of concrete in sea water at different locations on the U. S. coast and at Pearl Harbor, Hawaii, are being carried out by the U. N. Navy Department, Bureau of Yards and Docks. These investigations include tests on concrete from portland and high alumina cements, using different aggregates, admixtures, accelerators, protective coatings, etc.

Further details of the many investigations under way by various research agencies are given in Part I of Appendix A of this report.

Researches Under Way.—In order to secure information as to the character and extent of research in concrete throughout the United States and Canada the committee submitted two questionnaires to approximately 220 educational institutions, state, government, municipal, railway and other testing laboratories which the committee thought might be engaged in research on concrete and related subjects. These questionnaires were designed to secure information on:

(1) Researches recently completed, under way or planned for immediate future on concrete materials and plain and reinforced concrete.

(2) Subjects in the field of concrete materials and plain and reinforced concrete on which research is needed.

The condensed replies to these questionnaires assembled by subject, form Appendix A of this report. Part I of Appendix A gives the researches under way and Part II gives subjects on which research is needed.

A study of the investigations listed in Part I of Appendix A shows that at present considerably more attention is being given to plain concrete and concrete materials than to reinforced concrete. Portland cement and the high alumina cements are receiving much attention. New methods of testing cement are being studied. Investigations are under way to devise methods of testing aggregates which will give definite information as to their suitability for concrete for various purposes. There are numerous investigations on the effect of impurities in aggregates and grading of aggregates on the strength and other properties of concrete.

Investigations in the field of plain concrete cover a variety of subjects, the more important of which may be classified as follows:

Absorption
Admixtures
  1. Accelerators
  2. Deleterious Substances
  3. Inert
Age
Aggregates
Coloring
Consistency (Workability)
Curing
Dams
Durability
Elasticity
Expansion and Contraction
Extensibility
Fatigue
Field Control
Flow (Plastic)
Freezing and Thawing
Impact
Jigging and Vibration
Pavements
  1. Cracking
  2. Shrinkage
  3. Smoothness
  4. Value of Reinforcing
Poisson's Ratio
Pressure
Proportions
Sea Water
Shrinkage
Strength
  1. Compressive
  2. Flexural
  3. Tensile
Sulphate Soils and Waters
Temperature
Test Methods
Waterproofing and Protective Treatments

Investigations under way on reinforced concrete include such subjects as arches, beams, chimneys, columns, piling, pipe and slabs.

Suggested Subjects for Research.—Part II of Appendix A lists a number of subjects on which research is needed. This list was compiled from the answers to the committee's questionnaires and gives only the more important of the subjects suggested.

It is evident that there are a great many questions on which information is needed and on which research should be carried out as soon as possible. In cases where investigators have made tests on the subjects listed the committee will suggest to them that they prepare papers on the work in order that the information gained will be available to those interested.

The committee believes that there are a number of laboratories equipped to do research work in concrete, which would be willing to undertake studies along certain lines. Part of the work of the committee for next year will be to interest and aid such laboratories in carrying out investigations on which information is needed. A few laboratories have already made known their willingness to co-operate in this work.

Recent Reports on Researches in Concrete.—In order to inform the members of the Institute as to published papers and reports giving information on recent researches on concrete, reinforced concrete and related questions, the committee made a survey of the literature of the subject and compiled a list of the more important articles which is given in Appendix B of this report. Part I of Appendix B contains references to recent reports on researches carried out in the United States and Canada, and Part II gives references to reports on researches carried out in foreign countries. In compiling this list the committee has, in general, referred only to reports which were published during the past year. However, in a few cases, reports on foreign researches dating back as far as 1920 have been included. Papers published in the *Proceedings* of the Institute were not included. Attention is called to a recent bulletin of Purdue University on "Researches in Concrete," by W. K. Hatt, which will be of particular interest to the members of the Institute.

The committee wishes to express its appreciation to the various research organizations and individuals who have co-operated with it by furnishing information for this report.

H. F. GONNERMAN, *Chairman;*
F. E. RICHART, *Secretary.*

APPENDIX A.

PART I—INVESTIGATIONS IN THE UNITED STATES AND CANADA RECENTLY COMPLETED, UNDER WAY OR PLANNED FOR IMMEDIATE FUTURE ON CONCRETE MATERIALS, PLAIN AND REINFORCED CONCRETE.

Note: Since the majority of the investigations listed below have not been completed, reports giving data of the tests are, in general, not available. If a progress or final report has been published on an investigation, reference is made to the place of publication. The conclusions from certain of the researches listed are those given by the investigator and do not necessarily represent the views of the committee.

Investigations on Concrete Materials.

CEMENT:

Constitution of Portland Cement.—(Portland Cement Association Fellowship, U. S. Bureau of Standards, Washington)

An exhaustive investigation to determine what chemical compounds exist in portland cement; how the composition and character of raw materials, composition of raw mixtures, and degree of burning control their formation; what reactions take place when the cement is mixed with water and how these reactions are affected by foreign materials. Cement will be studied by means of the X-ray. Considerable work has been done on a study of the lime-silica-alumina system. The data obtained confirm the work of the Geophysical Laboratory which indicated that the major constituents are tricalcium silicate, dicalcium silicate, and tricalcium aluminate. The lime-iron oxide system is now being studied. Equipment is in use for producing the pure compounds in large quantities under closely controlled conditions. A very thorough study is being made of available petrographic methods looking toward developing a more precise one. Much attention has been paid to rechecking all data on the optical constants of the compounds that are being studied.

Preliminary studies have been made on the hydration of cement in the presence of excess water. There has been determined the hydrogen ion content of the solution at definite times, the rate of the separation of lime or alumina, and the nature of the sulphate and chloride formed by the aluminates in the presence of certain sulphates and chlorides. Some work has been done on the measurement of the heat developed by cement in setting and of the consistency of cement pastes and slurries. No publications have yet been issued.

Fineness Investigation.—(U. S. Bureau of Standards, Washington)

(a) Standard sieve series.
(b) Fineness of cement by air analyzer.

A Study of Bin-Storage of Portland Cement.—(Structural Materials Research Laboratory, Lewis Institute, Chicago)

Strength tests of concrete and mortar at ages of 7 days to 2 years on samples taken at intervals up to 3 years from cement stored as follows:

Cement stored at mill

(a) Mill bin (samples from different depths).
(b) Cloth sacks outdoors under canvas tarpaulin.

Cement stored at Lewis Institute

(a) Wood barrels in shed in yard.
(b) Cloth sacks in shed in yard.
(c) Cloth sacks in basement.
(d) Air-tight metal cans.

Relation Between the Strength of Cement Concrete in Which Cements Showing Widely Different Briquette Strengths are Used.—(U. S. Bureau of Public Roads, Washington)

Tests to determine the relation between the quality of concrete as determined by crushing strength and the quality of cement used in the concrete as determined by the conventional laboratory tests.

Studies of Methods of Molding Standard Briquets for Cement Tests. (Structural Materials Research Laboratory, Lewis Institute, Chicago)

A study of different methods of molding briquets to determine the effect of the recommendations of the "Manual of Cement Testing" prepared by A. S. T. M. Committee C-1 on Cement on the 7- and 28-day tensile strength of 1 : 5 standard sand mortar briquets.

Tests of Standard Sand Mortars.—(Structural Materials Research Laboratory, Lewis Institute, Chicago)

Tests made in co-operation with A. S. T. M. Committee C-1 on Cement to determine the form of specimen, mixture, consistency, method of molding, condition of curing, and age at test best adapted for an acceptable strength test of portland cement. The investigation included compression tests of 2-in. cubes and of 2 x 2-in. and 2 x 4-in. cylinders and tension tests of briquets made from standard sand mortars of different mixtures and consistencies; also compression tests of 6 x 12-in. cylinders of 1 : 3 concrete. Age at test 3 days to 1 yr. Effect of length of storage of briquets in moist closet and the 3-day strength of mortar and concrete specimens after exposure to steam at atmospheric pressure for 5 hr. also studied. Similar tests are being carried out in a number of other laboratories using portland cement from the same lot.

Tests of Mortars Made with Washed Standard Sand.—(Structural Materials Research Laboratory, Lewis Institute, Chicago)

Strength tests of mortars made from standard sand from which the dust had been removed by various chemical and physical treatments for the purpose of determining whether irregularities in cement testing are due to a coating of dust on the particles of standard Ottawa sand. The results indicate that a coating of dust on the standard sand is of comparatively little importance in causing irregularities in cement testing.

Comparison of Methods of Compacting Mortar Cylinders.—(Michigan State Highway Department, Ann Arbor)

Study of strength variations using different methods of compacting mortar.

Study of Actual Cross-Section of Briquets.—(Michigan State Highway Department, Ann Arbor)

To determine whether or not there is sufficient variation in the cross-section of briquets made by standard laboratory methods to influence the results of tests in any marked degree.

Effect of Height of Operators' Table on Strength of Briquets.—(Michigan State Highway Department, Ann Arbor)

Study of variation in strength of briquets molded on tables of different heights.

New Cements and Standard Methods of Testing.—(U. S. Bureau of Standards, Washington)

(a) Standard methods of cement testing.

(b) Procedure for comparative cement tests.

A Correlation Study of the Interrelationship of Individual Breaking Strengths of Portland Cement Mortars.—(Maine Technology Experiment Station, Orono)

A statistical analysis of laboratory data on 711 different samples of cement to establish basic or foundation data for further study of tension tests on portland cement mortars. Published as Bulletin 14.

Tests of Concrete Made with Cement that Failed to Meet A. S. T. M. Standard Specifications for Tensile Strength.—(Michigan State Highway Department, Ann Arbor)

Compressive strength, wear and modulus of rupture compared with that of specimens made with cement which passed the specifications.

A Study of Lumnite Cement.—(University of Washington, Seattle)

Tests of 2 x 4-in. mortar cylinders to determine equations for water-cement ratio strength curves and strength of mortar at different ages. Also tests to determine modulus of elasticity of concrete. Water and cement in different ratios added in form of paste to constant amount of aggregate. Strength determinations for each hour up to 24 hours then by days up to 28 days. Water-ratio-strength curves of same form as for portland cement. Strength of lumnite cement mortar begins at 6 hours.

Study of the Properties of Lumnite Cement and of Mixtures of Portland and Lumnite Cements.—(Missouri School of Mines, Rollo)

Determination of the properties of lumnite cement concrete and an investigation of the possibility of attaining similar results more economically with mixtures of portland and lumnite cements.

Effects of Low Temperature on the Setting and Strength of Lumnite Cement.—(Michigan State College, Lansing)

Compression specimens were exposed to cold weather of different degrees at various ages and with different coverings. Tests completed and report nearly ready for printer.

Effect of Tannic Acid on Alumina Cement Concrete.—(Yale University, New Haven)

Short-Time Tests of Lumnite Cement Mortars.—(Lehigh University, Bethlehem)

Strength tests at early ages.

Tests of High Alumina Cement.—(Iowa State Highway Commission, Ames) A study of the physical properties of high alumina cement.

Tests of Lumnite Cement.—(Wisconsin Highway Commission, Madison) Tests to determine its characteristics and to compare its properties with portland cement.

AGGREGATE:

Investigation of Local Materials for Use in Concrete.—(Tulane University, New Orleans)

Tests to determine the adaptability of local materials for making concrete of predetermined strength. To date about 150 cylinders and the usual tests of the materials have been made.

Investigation of Abrasion Test for Gravel.—Illinois Department of Public Works and Buildings, Springfield)

Comparison of results using plain and slotted barrels with and without abrasive charge. General survey by both methods on all Illinois gravels.

Study of Deval Abrasion Tests of Gravel.—(University of Michigan, Ann Arbor)

Tests to determine proper specification limits and methods as applied to Michigan gravels.

Statistical Study of the Size-Strength Relationship of Natural Sands in the Standard Tension and Compression Tests.—(Maine Technology Experiment Station, Orono)

Correlation or statistical analysis of data upon many natural sands to establish basic data on the effect of size and grading of natural sands upon their strengths in tension and in compression.

A Study of Laboratory Methods for Determining the Quality of Fine Aggregate for Use in Cement Concrete.—(U. S. Bureau of Public Roads, Washington)

Tests for hardness and abrasive resistance of sand grains and tests for durability of sand grains by means of an accelerated soundness test.

Study of Effect of Deleterious Matter in Sand and Gravel.—(Michigan State Highway Department, Ann Arbor)

Tests to determine whether or not specification standards for deleterious matter in sand and gravel are sufficiently rigid.

Screenings as Fine Aggregate.—(University of Kentucky, Lexington)

A study of the use of screenings as fine aggregate. Specimens made from screenings as received, after washing, and after washing and grading. Specimens tested at 28 days and after exposure to weather for 1 year.

Stone Screenings as Fine Aggregate.—(Illinois Department of Public Works and Buildings, Springfield)

Compressive and transverse tests of specimens using stone screenings as fine aggregate. General conclusion is that clean stone screenings are suitable; deleterious effect from high percentages of dust.

Tests of Poorly Graded Sands.—(Iowa State Highway Commission, Ames)

A study to devise methods of using poorly graded sands to produce concrete equivalent in workability and strength to that from normal materials. The mortar-void theory of proportioning was tried using briquets and 2 x 4-in. cylinders.

The Function of Coarse Aggregate in Concrete.—(University of Colorado, Boulder)

Study of effect of size, shape and kind of aggregate.

Studies of Laboratory Tests for Determining the Quality of Coarse Aggregate for Use in Cement Concrete.—(U. S. Bureau of Public Roads, Washington)

A continuing investigation including a study of various methods for determining the abrasive resistance of aggregates as well as a study of various forms of accelerated soundness tests, such as the sodium sulphate test, sodium chloride test and actual freezing and thawing tests.

Effect of Shale in Concrete Aggregates.—(Iowa State Highway Commission, Ames)

A study of the effect of varying amounts of shale in aggregates upon the compressive strength of 6 x 12-in. concrete cylinders and the flexural strength of 4 x 8 x 30-in. concrete beams.

Coated Stone as Coarse Aggregate.—(University of Kentucky, Lexington)

A study of the effect of coated or dirty stone on strength of concrete. A comparison of strength of concrete made from washed, unwashed, and dirty stone after exposure to the weather for 28 days, 1 and 2 years.

Effect of Stone Dust on Strength of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Tentative conclusion is that amounts of dust under 4 per cent (by weight of coarse aggregate) act as filler; increasing percentages cause decrease in strength. Effect greater on compressive strength.

Effect of Dust Coating on Stone.—(Michigan State Highway Department, Ann Arbor)

Study of variation of strength of concrete using different classes of dust coated stone.

Novaculite as Coarse Aggregate.—(Illinois Department of Public Works and Buildings, Springfield)
Transverse and compression tests of specimens containing novaculite (a weathered chert from Southern Illinois containing large percentages of clay in natural state). General results unfavorable to novaculite.
Sandstone as Aggregate for Concrete.—(University of Kentucky, Lexington)
Tests to determine the suitability of sandstone from Eastern Kentucky as aggregate for concrete. Compression tests of 6 x 12-in. cylinders, flexural tests of 7 x 7-in. x 4-ft. 3-in. reinforced-concrete beams and determination of physical properties of stone. Provided the wear does not exceed 12 per cent, sandstone can be used for structural work as both fine and coarse aggregate if properly prepared and proportioned.
Soft Stone as Coarse Aggregate.—(Illinois Department of Public Works and Buildings, Springfield)
Tests on stone of varying wear values. Some soft stones give superior strength due to better bond and absorption. General conclusion is that percentage of wear is not an indication of strength of concrete.
Zinc Chats as Aggregate for Concrete.—(University of Tennessee, Knoxville)
Tests to determine the suitability of chats as fine and coarse aggregate for concrete. Chats satisfactory for concrete aggregate when intelligently used.
Soundness Test of Rock for Coarse Aggregate.—(Wisconsin Highway Commission, Madison)
Soundness Tests of Stone.—(Minnesota Highway Department, St. Paul)
Soundness tests of various Minnesota stones using saturated solution of sodium sulphate. Fineness modulus of aggregate before and after test used as a measure of disintegration. Stone which showed disintegration in old ledges in quarry gave corresponding results in soundness test.

WATER:

Tests of Water for Mixing Concrete.—(Iowa State Highway Commission, Ames)
A study to determine the effect of the various chemicals found in the natural waters of Iowa on the strength of mortar briquets and cylinders.

Investigations on Plain Concrete.

ABRASION:

Abrasion Tests on Mortars.—(Iowa State Highway Commission, Ames)
A study of standard Deval abrasion tests on the mortar portion of the various concrete mixes used in highway work.

ABSORPTION:

Absorption Tests of Concrete at Different Ages.—(Michigan State Highway Department, Ann Arbor)
Determination of capacity of concrete to take up moisture at different ages.

ACCELERATORS: (see Admixtures)

ADMIXTURES:

Effect of "Cal" on Certain Properties of Cement.—(Drexel Institute, Philadelphia)
Tests on 5 domestic brands of cement, 1 foreign and a blended cement using admixtures of 0, 3, 5 and 8 per cent of "Cal." Storage temperatures 70, 55, 43 and 32 deg. F. Tests included determination of time of set, strength of neat cement in tension, strength of cement mortar in tension, compressive strength of machine-mixed concrete, modulus of elasticity and flowability. Relative effect of air and damp curing and effect of "Cal" on lumnite cement studied.
Effect of Admixtures of Calcium Chloride on Transverse Strength of Concrete.—(Dow Chemical Co., Midland, Mich.)
A study of the influence of varying amounts of calcium chloride when used as an admixture on the transverse strength of concrete of good quality placed and cured at low and at normal temperatures. Tests made on 6 x 12 x 36-in. concrete beams at ages up to 1 year. Six different brands of cement used.
Efficiency of Calcium Chloride as an Accelerator and Curative Agent when Used as an Admixture.—(Pennsylvania Highway Department, Harrisburg)
Tests to determine whether calcium chloride when used as an admixture in concrete can replace the usual curing method, reduce the curing time and accelerate the hardening of concrete road slabs. Concrete for specimens mixed in a 1-bag mixer. Transverse and compressive strength tests made at 7, 14 and 28 days, and 3 months.

Diatomaceous Earth as an Admixture for Concrete.—(University of California, Berkeley)

A study of the effect of diatomaceous earth on the strength and other properties of concrete.

Effect of Mixing Diatomaceous Earth with Cement.—(University of Idaho, Moscow)

Tension tests of 1 : 3 standard Ottawa sand briquets at 7 and 28 days in which the cement was replaced with 0 to 40 per cent by weight of diatomaceous earth. Data of tests published in *The Idaho Engineer* for December, 1925.

Effect of Addition of Celite to Limestone Screenings.—(Virginia State Highway Commission, Richmond)

Compression and absorption tests of 2 x 4-in. cylinders of 1 : 3 mortar made from limestone screenings with addition of celite up to 6 per cent. Tests made on 3 samples of screenings which contained different amounts of dust.

Effect of Small Additions of Soluble Silicates on the Setting of Portland Cement.—(Philadelphia Quartz Co., Philadelphia, Pa.)

Effect of Sugar on Tensile Strength of Cement Mortar.—(Michigan State Highway Department, Ann Arbor)

Tests of standard sand mortar briquets containing varying percentages of sugar.

Effect of Powdered Tufa in Concrete Exposed to Sulphate Waters.—(California Institute of Technology, Pasadena)

To determine whether additions of powdered tûfa to concrete mixtures will prevent disintegration of concrete in waters containing sodium sulphate and magnesium sulphate. 3 x 6-in. cylinders were alternately soaked in 4 per cent solution of an equal mixture of sodium sulphate and magnesium sulphate for 12 hours and then dried in air for 12 hours. After 6 months this process was discontinued and the specimens were left standing partly submerged in a concentrated solution of the same salts. Additions of finely ground tufa retard the disintegration of concrete in solutions of sodium and magnesium sulphate.

Study of Unit Weight of Powdered Materials.—(Structural Materials Research Laboratory, Chicago)

These tests are being made in co-operation with A. S. T. M. Committee C-9 on Concrete and Concrete Aggregates for the purpose of obtaining data on which to base a standard method for determining the unit of powdered materials which are used or likely to be used in concrete. The investigation included 5 methods, using hydrated lime, powdered talc, pulverized oil shale, "Cal," celite, gypsum and cements of different types.

AGE:

Effect of Age, Curing and Weathering on the Strength of Concrete.—(Structural Materials Research Laboratory, Chicago)

An investigation of the compressive strength of 6 x 12-in. cylinders and flexural strength of 7 x 10 x 38-in. concrete beams at ages of 28 days to 15 years cured under different conditions. In order to determine the effect of exposure to weather on strength at different seasons of the year, the making of part of the specimens was repeated in the months of April, July, October and December.

Long Time Tests of Concrete.—(University of Wisconsin, Madison)

Study of effect of age of air and of water storage on the crushing strength of concrete made of different cements and different aggregates. Tests of crushing strength and expansion for ages up to 2 years have been completed. Crushing tests made on 6 x 12-in. cylinders of 1 : 1½ : 3, 1 : 2 : 4 and 1 : 3 : 6 machine-mixed concrete of medium and sloppy consistency. Concrete made from 4 different grades of portland cement using crushed dolomite, crushed granite and gravel as coarse aggregate. Specimens cured in dry air of laboratory and out-of-doors. 1760 cylinders made for test over a period of 100 years. Includes auxiliary tests and expansion tests of concrete; also freezing and thawing tests at 28 days.

Effect of Age on Compressive Strength of Concrete.—(New Mexico State Highway Department, Las Cruces)

Studies of the effect of age on compressive strength of concrete of various mixtures and consistencies. Three mixtures were designed to represent 3 degrees of compressive strength at 28 days, viz., 1,500 lb., 2,500 lb., 3,000 lb. One hundred 6 x 12-in. cylinders are being made at various consistencies for each mixture and will be tested at ages of 7, 28 and 90 days.

AUTOGENOUS HEALING:

Autogenous Healing of Concrete.—(University of Colorado, Boulder)

An incidental investigation to obtain more of a quantitative measure of this phenomenon than seems to be now available.

COLORING:

Tests of Portland Cement Colors.—(Structural Materials Research Laboratory, Chicago)

These tests are intended to furnish information regarding the relations between the chemical composition of coloring materials and their permanence

in portland cement mortars, their tinting power and their effect on the strength and other qualities of cement mortars. Samples of 280 colors have been received from 35 manufacturers. The investigation will include chemical and physical tests on the colors and strength and exposure tests (on roof and in dark basement) on cement mortars in which the colors are used. A study will also be made of methods of incorporating the colors in the mortars.

CONSISTENCY:

Studies of Methods for Determining the Consistency of Cement Concrete and Mortar.—(U. S. Bureau of Public Roads, Washington)

Extensive laboratory and semi-field tests for the purpose of determining the most satisfactory method of measuring of consistency of portland cement concrete. Comparative tests have been made using the slump test, the flow test and a special device known as the *plate test* developed by the Bureau. All of these tests have been conducted with a view of developing a method which will control the water-cement ratio and therefore the strength of the concrete. See "A New Test for Consistency of Paving Concrete," by Jackson and Werner, p. 204, 1925, *Proceedings*, A. S. T. M.

A Study of the Relation between Consistency and Strength of Portland Cement Concrete.—(Texas A. & M. College, College Station)

A study of the relation between the strength of concrete of various mixes and consistencies as measured by several methods. Several mixes both of arbitrary proportions and of proportions conforming to design theories used with varying amounts of water in order to secure different consistencies. Cylinders tested for compressive strength. Consistency measured by means of slump cone, flow table and other devices.

Paste-Ratio as a Criterion of Workability of Concrete.—(Structural Materials Research Laboratory, Chicago)

The purpose of these tests is to study the effect of the amount of cement-water paste on the workability of mortar and concrete mixtures. Earlier tests have shown that there is a relation between these factors and that the workability is, in a degree, measured by the ratio between the absolute volume of the cement paste and the space in the concrete unoccupied by aggregate. The absorption of the water by the aggregate during the mixing period has an important effect upon the volume of paste produced. More than the usual care will be given in handling to prevent the loss of paste, and to the measurement of the volume of concrete produced for a given volume of aggregate. Supplementary tests will be made to determine the rate of absorption during mixing. Compression tests of 6 x 12-in. concrete and mortar cylinders for a variety of conditions will be made at 28 days.

CURING:

Calcium Chloride as a Curing Agent for Concrete.—(University of Michigan, Ann Arbor)

A study of the efficiency of calcium chloride as a curing agent for concrete pavements. Varying quantities of calcium chloride tried using cements of various degrees of seasoning; a few tests made using sodium chloride for curing.

Curing of Concrete with Calcium Chloride.—(Dow Chemical Co., Midland, Mich.)

An invetigation of the transverse strength and resistance to wear of 6 x 12 x 36-in. concrete beams cured with surface application of calcium chloride. Tests made on concrete of different consistencies using different gradings of aggregate and 3 different brands of cement.

Investigations in Curing of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Study of suitability of various materials as curing agents, when used as surface applications and as admixtures. Tests on mortar and concrete specimens at ages of 7 days to 2 years. Strength, wear, setting time, etc., determined.

Relative Efficiency of Various Periods of Water Curing.—(California Highway Commission, Sacramento)

A field study in which sections of pavement were cured with earth covering kept wet for varying periods. Cores drilled at various ages up to 1 year for test in compression.

Effect of Par-Lock Coating on Hydration of Concrete.—(Santa Fe R. R. and Massey Concrete Products Corporation)

Tests to determine whether this treatment will assist in retaining moisture in concrete to secure complete hydration of concrete in arid regions. Test cylinders coated and stored under laboratory conditions in Chicago and under field conditions at Barstow, Calif.; untreated cylinders made at same time and tested simultaneously. No appreciable effect of coating observed.

DAMS:

Experiments on a Large Test Dam.—(Committee of Engineering Foundation, New York)

Tests to be conducted on a proposed single arch type dam located 80 miles east of Fresno, Calif., on Stevenson Creek.

Dam to be first built to a height of 60 ft. and tested repeatedly under a variety of load and temperature conditions for about 1 year. After information on the 60-ft. dam has been obtained the dam will be raised in steps of 10 ft. each to a height of about 100 ft. The dam will ultimately be raised to a sufficient height to cause failure if practicable. Thickness of the proposed test dam is 7½ ft. at the base and 2 ft. above elevation 30. The span at the crest of the dam 60 ft. high is about 125 ft.

In order to determine the distribution of stresses in the proposed test dam special arrangements are being made to measure the strains, deflections and temperature changes at as many points as practicable. The tests will be conducted under a great variety of conditions as to loads and temperatures. At accessible positions the strains will be measured by strain gages or similar devices; in inacessible places electric tele-strain gages will be used. Temperature variations will be measured at over 100 points.

Extensive laboratory tests will also be made in order to determine the physical properties of the concrete in the test dam.

For tentative outline of the tests see *Engineering Journal* (Montreal) for December, 1925, p. 493.

DURABILITY:

Durability of Concrete.—(Hydro-Electric Power Commission, Toronto, Can.)

A study of the factors governing the resistance of concrete to the action of frost and other weathering agents with the object of learning how to make concretes having resistance to these agents in an economical manner. Includes study of absorption by capillary action in concretes and mortars, freezing and thawing tests on small and large specimens, microscopic studies of pore distribution in concretes using a wide variety of concretes and mortars.

Investigation of the Causes which bring about the Gradual Deterioration of Concrete.—(Columbia University, New York City)

This investigation includes a survey of the literature on the subject as well as numerous field inspections. A large number of samples of concrete have been secured from structures which show incipient disintegration and are being investigated microscopically.

ELASTICITY, MODULUS OF:

Investigation of the Modulus of Elasticity of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Observations made on corners of pavement slab under load, including deflections and strains in upper fibers at various points.

Modulus of Elasticity of Concrete.—(Iowa State Highway Commission, Ames)

An investigation of the effect of varying sand ratio, consistency and curing period upon the modulus of elasticity of concrete. Direct reading of deflections made with micrometer microscope on beam specimens.

ELECTROLYSIS:

Electrolysis in Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Set-up made on pavement slab using continual battery current in determining effect of calcium chloride on conductivity.

EXPANSION AND CONTRACTION:

Moisture-Absorption Expansion of Concrete.—(University of California, Berkeley)

Determination of the expansion and contraction of various concretes with change in moisture content and rate at which such expansion and contraction occurs; 3 x 3 x 40-in. specimens are kept in a room especially constructed for the tests where temperature and humidity may be held at any desired amount. Contraction and expansion from conditions of oven dry to full saturation have been determined and also the rates at which expansion takes place at various conditions of atmospheric moisture.

Expansion due to change in moisture content, other things remaining equal, varies with cement-ratio. For the same character of aggregate and for a given cement-ratio, expansion is a function of the surface modulus of the aggregate.

Properties of Plain Cement Mortars and Concrete.—(Department of Chemical Engineering, University of Michigan, Ann Arbor)

Studies of the properties of cement mortars and concrete considering mainly those factors which influence changes in volume, especially moisture and freezing. The influence of chemical composition of cement has also been studied.

Considerable unpublished data are on hand consisting of observations on the expansion and contraction of neat cement mortar specimens, some of which have been under observation at regular intervals for more than 20 years. Effect of high magnesia and high lime has been studied; also the self waterproofing of concrete with time.

Investigation of Expansion and Contraction of Mortars and Concrete.—(U. S. Bureau of Standards, Washington)

Determination of Coefficient of Expansion of Several Non-Metallic Structural Materials Including Portland Cement Mortars.—(Engineering Experiment Station, University of Nevada, Reno)

Changes in length of specimens about 30 in. long will be measured by means of a special apparatus.

Thermal Coefficient of Expansion of Concrete.—(Iowa State Highway Commission, Ames)

A study of the thermal coefficient of expansion of concrete made from different mixes.

EXTENSIBILITY:

Investigation of Extensibility of Concrete.—(Purdue University, Lafayette, Ind.)

A study of extensibility of concrete and means for improving. Extensibility does not seem to follow the laws of strength. Surface conditions are dependent upon curing and exposure. Mesh reinforcement is effective in postponing the appearance of the first eye-visible crack; it does not seem to postpone the appearance of crazes. Report made to committee on Structural Design of the Highway Research Board, National Research Council. See "Extensibility of Concrete," by W. K. Hatt, 1926 *Proceedings,* American Concrete Institute.

FATIGUE:

Fatigue of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Application of repeated load to cantilever beams in order to determine endurance limit of concrete. Critical percentage of load found to be approximately 50 to 55 per cent. Tests also made on static fatigue.

FIELD TESTS AND CONTROL:

Field Tests of Concrete.—(Structural Materials Research Laboratory, Chicago)

Field tests were carried out during the construction of a reinforced-concrete building in Chicago for the purpose of obtaining a record of the quality and the uniformity of the concrete and concrete materials going into the structure. The tests are of particular interest in view of the new form of concrete specification used. This specification was based on the fact that the compressive strength of concrete so far as proportions are concerned is controlled by the water-ratio of the concrete, that is the quantity of mixing water as compared with the quantity of cement. The different qualities of concrete were, therefore, specified by stating the quantity of water to be used with each bag of cement. No specific reference was made to the proportions of cement and aggregate other than by general clauses as to workability.

The tests gave valuable information on the uniformity of the concrete made under this type of specification. During the fall and winter months, data were obtained on the effect of low temperature on the strength of concrete. See "Use of Water-Ratio Specification in the Construction of Portland Cement Association Building," by McMillan and Walker, 1926 *Proceedings,* American Concrete Institute.

Control of Concrete by Water-Cement Ratio in Field Operations.—(Barney-Ahlers Construction Corp., New York City)

Study of method for economical control and production of concrete of uniform strength. Includes analysis of aggregates, tests for moisture cement, control of water-cement ratio. Tests made on various aggregates using same water-cement ratios. Apparatus used includes the "Ahlers" concrete strength regulator, field apparatus for determining moisture content of aggregate, etc. Water-cement ratio a success in field control; concrete should be specified by water-cement ratio. See "New Experiences in Concrete Control," by John G. Ahlers, 1926 *Proceedings,* American Concrete Institute.

An Investigation of the Accuracy of Field Methods of Proportioning.—(Illinois Department of Public Works and Buildings, Springfield)

Tests made at various pavement proportioning plants using a "Void Bulk Meter" apparatus for checking volume, gradation, voids, unit weight and compaction of aggregates during handling and bulking of fine aggregates. Determinations made on full size batches as proportioned by concrete plants.

FLOW:

Flow of Concrete Under Continued Stress.—(University of California, Berkeley)

A study of the plastic deformations which occur with time in concrete subjected to pure compressive stresses. Specimens are 6 x 24-in. cylinders. The desired compression is maintained by means of car springs. Deformations are measured with a Berry strain gage. Half of the specimens are kept in air dry

condition, remainder are kept completely saturated with water. Specimens kept in specially constructed rooms wheree the humidity and temperature can be maintained constant.

Plastic Flow of Concrete.—(Engineering Experiment Station, Ohio State University, Columbus)

Study of the amount of flow that occurs in concrete. Tests at various ages using different aggregate materials, mixes, consistencies and curing conditions. Deformation measurements made on 4 sides of 4 x 4-in. prisms using 20-in. Berry strain gage. Data so far available indicate that plastic flow occurs; that there is considerable effect from age; that the early flow is the important flow and that there is little short time flow after one year.

FRICTION, COEFFICIENT OF:

Coefficient of Friction between Earth and Concrete Slabs.—(Iowa State Highway Commission, Ames)

Determination of the coefficient of friction of a concrete slab moving from rest and in motion on various types of subgrade soil; 24 x 24 x 48-in. concrete slabs cast on ground and pulled with hoist; pull measured by spring dynamometer.

FREEZING AND THAWING:

Effect of Type of Concrete on Resistance to Repeated Frost Action.—(U. S. Bureau of Public Roads, Washington)

An extensive series of tests for the purpose of determining what relation, if any, exists between the resistance of concrete to repeated frost action and the various physical properties of the coarse aggregates used therein. Trap rock, limestone, sandstone, gravel, and blast furnace slags of widely varying characteristics were used.

GRADING OF AGGREGATE, EFFECT OF:

A Study of Concrete Made with Various Gradings of Sand and Gravel.—(Purdue University, Lafayette, Ind.)

A study of the relative value of fine and coarse sand with respect to strength and yield, taking into account the ordinary conditions and variations in moisture in the aggregate.

Effect of Varying the Quantity of Sand in Aggregate.—(Iowa State Highway Commission, Ames)

A study to determine the effect of varying ratios of fine and coarse aggregate in concrete mixes upon strength and resistance to wear; 6 x 12-in. cylinders, 4 x 8 x 30-in. beams and wear blocks were subjected to the usual tests. Specimens were made both in the field and in the laboratory.

Effect of Grading of Coarse Aggregate on Strength of Concrete.—(Iowa State Highway Commission, Ames)

A study of the effect of grading of coarse aggregate upon strength of concrete made from different proportions. Strength tests were made on 4 x 8 x 30-in. and 8 x 8 x 30-in. beams, and on 6 x 12-in. cylinders.

Gradation of Concrete Aggregates.—(Illinois Department of Public Works and Buildings, Springfield)

Tests to determine effect of various combinations on strength of concrete.

IMPACT:

A Comparison of Static and Impact Strains in Concrete.—(Johns Hopkins University in co-operation with U. S. Bureau of Public Roads)

Determination of impact modulus of rupture 6 x 8-in. plain concrete beams 220 days old loaded statically at the third points. Unit deformation measured at 6 levels on each side of beam, also deflection at center. Modulus of rupture determined by upward shift in neutral axis and by sharp change in direction of load-fibre strain curves. The process is repeated for impact loads using 5 different cushioning media under the falling weight in an attempt to vary the time duration of the load application. Apparatus consists of special impact testing machine. "Goldbeck" graphic strain gage for measuring unit deformations. 1/10,000 Ames dial for measuring deflections and a special coil spring accelerometer calibrated against time-space curves for measuring force of impact.

Impact Tests.—(Illinois Department of Public Works and Buildings, Springfield)

Impact loads have been calculated at varying wheel loads, speeds and height of drop or rise, in terms of equivalent static load.

JIGGING AND VIBRATION:

Effect of Jigging on the Strength of Concrete.—(University of California, Berkeley)

A study of the effect of jigging fresh concrete for various lengths of time. About 600 cylinders of various water-cement ratios and from various types of aggregates have been tested. Cylinders of fresh concrete are jigged on a table

for which the height of drop and rate of vibration may be regulated. Periods of jigging ranged from 5 minutes to 6 hours.

Effect of jigging depends almost entirely upon the gradation of the aggregate. With a well-graded aggregate, conforming closely to Fuller's ideal curve but with a small excess of fines, there is a consistent increase in strength with jigging up to about 3 hours. This is true regardless of the water-ratio, but the increase in strength is greater in dry than in wet mixtures. With a poorly graded aggregate the strength decreases with the time of jigging, and with the wet mixes a decided separation occurs after a very short period of jigging.

A Study of the Vibrolithic Process of Finishing Concrete Pavement Slabs.—(Iowa State Highway Commission, Ames)

An investigation to determine effect of the vibrolithic process as applied to the finishing of concrete pavement slabs. Ten 7 x 75 ft. x 8 in. concrete slabs built under as near field conditions as possible. Slabs divided into 750 beams, 12 in. x 8 in. x 7 ft. Transverse tests made on beams and compression tests on cores drilled from beams after transverse tests.

An Investigation of the Merits of the Vibrolithic Process in Concrete Road Construction.—(U. S. Bureau of Public Roads, Washington)

Tests to determine the relative transverse strength of concrete slabs of different proportions constructed in the ordinary manner as compared with slabs constructed by the vibrolithic process.

PAVEMENTS:

Condition Survey of Concrete Pavements with Simultaneous Soil Survey.—(Michigan State Highway Department, Lansing)

A study of the adequacy of pavement and underdrainage design; value of reinforcement; history of different brands of cement; methods of finishing; construction, etc. Investigation includes survey of 208 miles of pavement and 20 miles of subgrade. Actual surface of all pavements sketched; soils mapped according to U. S. Bureau of Soils nomenclature and all types of soils tested in laboratory. Information concerning each project classified for purpose of comparison.

Reinforcement is effective in producing concrete which is apparently stronger than plain concrete; disintegration after cracking is prevented by reinforcement. Gravel sub-base is very suitable with adequate drainage but is actually a detriment unless thoroughly underdrained.

Distribution of Load in a Pavement Slab.—(Illinois Department of Public Works and Buildings, Springfield)

Distribution of wheel load on pavement slab determined by subgrade pressure cell readings.

Survey to Determine the Value of Reinforcing in Concrete Pavement.—(Highway Research Board, National Research Council)

Pavements in good and bad condition were compared to determine the relative value of different types of reinforcement. In addition to differences in the subgrade conditions, notes were taken on the age, mix, traffic and cross-section variations. Complete report available about March, 1926. A summary of conclusions may be obtained from Highway Research Board, National Research Council, Washington, D. C.

Reinforcing Steel in Concrete Pavements.—(Iowa State Highway Commission, Ames)

A study of the behavior of reinforcing steel in concrete pavements; special attention directed to bond stress and the behavior of steel at cracks.

Survey of Pavements to Obtain Data on Shrinkage of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Comparison of effectiveness of different methods of curing concrete pavements by survey of the number of shrinkage cracks with form after the pavement has been in use approximately 1 year. The indications are that pavement cured with calcium chloride shrinks much less than that cured with earth and water.

Investigation of the Cause and Control of Haircracks in Concrete Pavements.—(Iowa State Highway Commission, Ames)

Study of field conditions probably causing haircracks and the duplication of these conditions using small-size slabs. Cracks produced almost at will. Portion of work published in *Public Roads,* August, 1925, under title "Tar Paper on Loess Subgrade Lessen Haircracks in Concrete Pavements," by R. W. Crum.

Working of Pavements.—(Illinois Department of Public Works and Buildings, Springfield)

Experiments to determine the nature and extent of this phenomenon included observations over various temperature cycles. Movement at various points on the pavement slab was recorded.

Investigation of Smoothness of Concrete Pavement.—(Illinois Department of Public Works and Buildings, Springfield)

Survey made on pavement completed in various years with different types of finishing machines under different conditions of grade. Records of smoothness obtained by profilometer designed by Illinois Highway Laboratory which is capable of measuring depression or elevations of pavement surface within 1/20

of an inch. Various methods of checking surfaces with straight edge and other apparatus compared with profilometer record.

POISSON'S RATIO:

Poisson's Ratio for Concrete.—(University of California, Berkeley)

Determination of the elastic properties of concrete under axial compression and effect of age on these properties. Axial compression and lateral expansion of 6 x 12-in. cylinders measured by the optical lever principle; methods employed are a modification of those reported by A. N. Johnson in the 1924 A. S. T. M. *Proceedings.* Stress-strain curves for both lateral and axial deformations are curved lines practically from the point of beginning. The indications are that the stress-strain relation becomes more nearly a straight line as the age increases. Poisson's ratio varied from about 0.12 to 0.22 with an average value of 0.15. There is no indication that age or strength has any appreciable effect on Poisson's ratio.

PROPORTIONS:

Economics of Concrete Mixtures.—(Hydro-Electric Power Commission, Toronto, Canada)

Development of data whereby the probable best proportions for any given set of aggregates can be approximated closely in advance and also whereby the effect on cost of screening, regrading, or other changes in the aggregates can be determined so that their advisability may be judged. The same data because of its completeness will be available for many other studies of concrete mixtures. Tests cover a wide range of mixtures having different cement contents, consistencies and gradations of aggregates.

Analysis of Cured Concrete.—(Iowa State Highway Commission, Ames)

A study of various methods for determining the cement content and the proportions of aggregate in cured concrete. Mechanical separation, chemical analysis and microscopic observation and measurement were used on prepared specimens wherein all the constituents were accurately known.

Determination of Proportions of Hardened Concrete.—(U. S. Bureau of Public Roads, Washington)

This investigation includes a limited study of the possible methods of determining the proportions of hardened concrete from chemical analysis.

Application of Chemical Analysis to Determination of Causes of Defective Concrete.—(Dow Chemical Co., Midland, Mich.)

A study to develop and apply methods for chemical analysis of concrete which will aid in determining the conditions prevailing at time of installation. Analyses made on samples of known proportions as well as on field specimens.

Proportioning Concrete.—(Washington University, St. Louis)

Twenty-eight day compression tests on 6 x 12-in. cylinders. Concrete proportioned by theories advanced by Edwards, Abrams and other investigators. Principal purpose to study the application of suggested methods for scientific proportioning to materials common to the St. Louis district.

A Study of Abrams' Method of Proportioning.—(Carnegie Institute of Technology, Pittsburgh)

Application of Abrams' theory of proportioning to Pittsburgh aggregates. Proportions determined for 2,000- and 3,000-lb. concrete using Allegheny River gravel and fine and coarse Allegheny River sands. Study was also made of effect of height of drop of aggregates on their weight per cu. ft. in order to determine relation between the rodded volume and the volume in the batches. 1,670 lb. and 2,840 lb. per sq. in. obtained for concrete designed to have compressive strength of 2,000 and 3,000 lb. per sq. in., respectively.

Tests of Miscellaneous Aggregates in Concrete of Constant Water-Cement Ratio.—(Structural Materials Research Laboratory, Chicago)

Tests made using a variety of fine and coarse aggregates in order to determine the feasibility of specifying proportions for concrete in terms of water-cement ratio and workability only, allowing the quantities of aggregates to be controlled by these factors.

PRESSURE:

Effect of Pressure During Setting on Strength of Concrete and Cement.—(Engineering Experiment Station, Virginia Polytechnic Institute, Blacksburg)

Tests to determine strength and porosity due to pressure during setting. Specimens subjected to hydraulic pressure in pressure drum about 12 in. in diameter and 3 ft. long; none of mixing water squeezed out. Specimens placed in polished brass tubes and sealed off at each end by leather cups.

Effect of Pressure on Setting of Concrete.—(University of Iowa, Iowa City)

Determination of the possible difference in strength which may exist in concrete poured in high columns. Concrete allowed to set under various pressures applied in testing machine.

SEA WATER:

Tests of Concrete in Sea Water.—(Navy Department, Bureau of Yards and Docks, Washington)

Determination of effects of sea water on concrete using various kinds of cement, stone and gravel, various mixtures and various methods of surface treatment of concrete. Tests being carried out at Portsmouth, N. H. (similar tests under way at Pearl Harbor, T. H.). Thirty-five specimens 14 x 14 in. by 13 ft. were made; 20 completed in fall of 1924, remainder in July and August, 1925. All specimens hung in place in salt water by Oct. 1, 1925. Observations to be made on specimens every 6 months for a period of 15 to 20 years. Complete details concerning manufacture of specimens recorded. Purpose of tests is to determine the effect of sea water on various kinds of concrete from high alumina cement (Atlas lumnite), French alumina (fondu cement) and portland cement. Factors studied include effect of mix, quantity of water, length of time of immersion in sea water; effect of using crushed stone and gravel as aggregate; effect of celite and trass as lubricators and of calcium chloride as an accelerator; also the effect of sea water on various specimens which have been given different surface treatments for protection.

Standard concrete cylinders were made from all specimens and tested at various ages and considerable data are available as to the comparative strengths. The behavior of various specimens during setting, especially those from lumnite and French alumina cements has been observed. In general, the data which are now available are preliminary and deal more with the details and descriptions followed in making the specimens. No real data as regards the effect of sea water will be available until observations have been made for some years to come.

Tests of High Alumina Cement Concrete in Sea Water.—(Navy Department, Bureau of Yards and Docks, Washington)

Actual service tests over a term of years of high alumina cement concrete in sea water. Structures have been built and specimens placed. Reports of conditions will be submitted from time to time.

Portsmouth, N. H.—Test specimens of high alumina cement concrete immersed in sea water.

Hampton Roads, Va.—Seaplane runway composed of precast piles and poured in place deck, all of high alumina cement concrete.

Key West, Fla.—High alumina cement concrete sheet piles and some cement gun protective covering of high alumina cement concrete on portland cement concrete piles and underside of slab of quay wall, all exposed to sea water.

Mare Island Navy Yard, Calif.—Various jobs around the navy yard, some in sea water and some on shore.

Puget Sound, Wash.—Precast lumnite cement concrete shells for cylinders supporting fitting-out pier. These shells are being filled with plain portland cement concrete after placing. This is understood to be the largest job yet undertaken with high alumina cement in this country.

Pearl Harbor, T. H.—Test specimens of high alumina cement concrete immersed in sea water.

No conclusions drawn as to durability of high alumina cement concrete in sea water. Work done to date indicates need of more careful workmanship, better water control than is the case with portland cement concrete.

Tests of Protective Coatings for Concrete Piles in Sea Water.—(Navy Department, Bureau of Yards and Docks)

Tests at Charleston, S. C., to determine the efficacy of various kinds of protective coverings. Piles when well dried out and during loading for driving were painted over the extreme tidal range with 2 coats of wax tailings. Other piles were coated with "Tarvia A" undiluted, applied hot or with standard Mexican asphalt macadam binder, 60 lb. being mixed with 10 qt. of petroleum spirits and the mixture applied hot. A fourth treatment consisted of 25 lb. of asphalt mixed with 10 qt. of mineral spirits applied cold. Over this primer was applied hot undiluted asphalt.

Tests of Protective Coatings on High Alumina Cement Concrete in Sea Water.—(Navy Department, Bureau of Yards and Docks)

Tests over a term of years to determine the efficacy of various kinds of protective coatings on high alumina cement concrete in sea water. Specimens of 1:2:3½ lumnite cement concrete coated 24 hr. after making as follows have been placed in sea water in Puget Sound, Wash., at various depths:

(a) A mixture of 50 per cent petrolastic cement and 50 per cent gasoline sprayed on cold by means of an air gun.
(b) "Hydroproof," a water asphalt solution.
(c) Water gas tar applied cold.
(d) "Tarvia A" applied hot.
(e) "Gilso" roofing cement applied cold.
(f) Roofing pitch coal tar applied hot.
(g) Wax tailings applied hot.
(h) Rosin coal-oil solution applied cold.
(i) Two-coat work consisting of water gas tar applied cold followed by "Tarvia A" applied hot.

SHRINKAGE:

Shrinkage of Concrete.—(University of Minnesota, Minneapolis)

A study of the factors controlling the shrinkage and expansion of concrete under service conditions. Includes the manufacture and observation with strain gages of various concrete laboratory specimens as well as a study of bridges, buildings and chimneys in service considering such factors as aggregates, cement, water, curing, moisture, admixtures, steam, etc. Shrinkage may be expected in all concrete structures built in the usual manner of standard materials and cured normally; expansion may occur due to moisture, etc.

STRENGTH, COMPRESSIVE:

Comparison of Drilled and Cast Cylinders.—(Illinois Department of Public Works and Buildings, Springfield)

A tabulation of data from compression tests on both types of specimen shows higher strengths for drilled specimens. Considerable variation in strengths found.

Effect of Addition of Cement to 1:2:4 Concrete.—(Virginia State Highway Commission, Richmond)

A study of the strength of a 1:2:4 mix upon adding different percentages of cement using gravel and crushed stone as coarse aggregate; 3 per cent celite used as an admixture in a few tests.

General Concrete Investigation.—(U. S. Bureau of Standards, Washington)

A study of the effect of certain factors on the strength of concrete at different ages.

High Early Strength Concrete from Portland Cement.—(Structural Materials Research Laboratory, Chicago)

A study of the compressive strength of 6 x 12-in. portland cement concrete cylinders at ages of 1, 3, 7 and 28 days, 3 mo. and 1 yr., when the following factors, singly and in combination were varied:

(1) Quantity of mixing water.
(2) Quantity of cement.
(3) Condition of curing.
(4) Time of mixing concrete.
(5) Admixture of 2 per cent of calcium chloride.

Properties of Plain Concrete.—(University of Illinois, Urbana)

Studies of the properties of plain concrete and of the effect of various elements entering into the results. This is a continuation of the work reported in Bulletin 137 of the Engineering Experiment Station.

Investigation of the Variation in Strength of Concrete.—(Carnegie Institute of Technology, Pittsburgh)

A study of the variations in the strength of concrete with every factor influencing strength kept constant, with ultimate object of determining strength dispersion as a function of the size of test specimen (6 sizes used varying from 1 to 6 in. in diameter), age of concrete, grading of aggregate, ratio of maximum size of aggregate to diameter of test cylinder, etc. The variation in compressive strength of concrete or mortar is obtained by making and testing a number of multiplicate test cylinders taking every care to insure equal fabrication and test treatment. Data studied by means of the mathematical theory of statistics.

The compressive strength dispersion for a given *concrete* (mix) was found to be a function of the diametrical size of the test specimen and to be also dependent upon the size of the largest aggregate in the mix. The safe working stresses for concrete, as have been shown to be dependent upon the strength dispersion, are shown to vary with the specimen diameter and to be usually higher than those accepted by the codes.

Strength and Modulus of Elasticity of Concrete.—(Engineering Experiment Station, Iowa State College, Ames)

A study of the compressive, tensile, and flexural strength of concrete and the modulus of elasticity in tension and compression.

STRENGTH, FLEXURAL:

A Comparison of Transverse and Compressive Strength.—(Illinois Department of Public Works and Buildings, Springfield)

This study includes tests on compression specimens drilled from beams which had been tested for transverse strength. Both gravel and stone aggregates with various gradations within the Illinois Highway Specifications were used. See "Transverse Testing of Concrete" by Clemmer and Burggraf, 1926 *Proceedings,* American Concrete Institute.

Transverse Tests of Concrete Beams of Different Sizes.—(Pennsylvania Highway Department, Harrisburg)

Tests to standardize a method of making transverse tests of concrete. Beams of different depths and widths tested as cantilever and as simple beams at ages of 14 days to 3 months and moduli of rupture compared.

Modulus of Rupture of Concrete Beams.—(Iowa State Highway Commission, Ames)

A study of the effect of size and shape of test beam upon the modulus of rupture; 4 x 8 x 30-in. and 8 x 4 x 60-in. beams tested on different span lengths.

Studies of Method of Loading in Flexural Tests of Concrete.—(Structural Materials Research Laboratory, Chicago)

These tests are being carried out in co-operation with A. S. T. M. Committee C-9 on Concrete and Concrete Aggregates, and consist of a study of methods of making flexural tests of concrete in order to determine the size and shape of beam and the method of loading which will give most uniform and satisfactory results. The investigation covers:

(1) Study of effect of varying ratio of depth to width and span on modulus of rupture;
(2) Comparison of three methods of loading on modulus of rupture: (a) center load, (b) 1/3 point load, (c) cantilever load;
(3) Study of effect on modulus of rupture of varying distances between support and thrust bearing in cantilever beam test;
(4) Parallel compression tests of concrete cylinders.

STRENGTH, TENSILE:

Investigation of Tensile Strength of Concrete.—(University of Maryland, College Park)

Study of the tensile strength and elastic behavior of concrete under tension using special tension specimens and apparatus, lateral and longitudinal deformations in concrete measured by means of special mirror apparatus.

Tension Tests of Concrete Cylinders.—(Structural Materials Research Laboratory, Chicago)

An investigation to determine the strength of concrete in direct tension, using 2, 6 and 10 in. diameter cylinders 3 diameters long, and 6 in. diameter cylinders 12, 18 and 30 in. long. Special tension grips used for holding specimen during test. Among the factors studied are: (1) size, grading and type of aggregate; (2) quantity of cement; (3) quantity of mixing water; (4) age of concrete; (5) curing of concrete; (6) area in tension; (7) length of specimen.

The investigation includes a study of the relation of the strength of concrete in direct tension to compressive strength of 6 x 12-in. concrete cylinders and flexural strength of 7 x 10 x 38-in. concrete beams for a variety of conditions of test.

SULPHATE SOILS AND WATERS:

Tests of Concrete Exposed to Sulphate Soils and Waters.—(Structural Materials Research Laboratory, Chicago)

This investigation was begun in 1921 and includes field tests on 10 x 24-in. concrete cylinders, 1,000 each exposed to sulphate soil at Montrose, Colo., and to sulphate water at Medicine Lake, South Dakota, and a smaller number at 3 different sites in western Canada. About 6,000 4 x 8-in. concrete cylinders exposed to various sulphate solutions at Lewis Institute, Chicago. Studies are being made of the effect of the following factors on the resistance of concrete to attack by sulphate soils and waters; consistency of concrete, size and grading of aggregate, type of aggregate, quantity and composition of cement, powdered admixtures, integral water and alkali-proofing compounds, surface coatings, age and curing conditions of concrete. A comparison of results of field and laboratory tests is being made.

Investigations of Concrete Exposed to Alkali and Peat Soils and to Alkali Water.—(University of Minnesota, St. Paul, in co-operation with Minnesota Department of Drainage and Waters and the U. S. Bureau of Roads)

Studies and tests of experimental cylinders of many different types of concrete exposed to artificial sulphate solutions in the laboratory. Experimental drain tile and cylinders have been made and exposed to various field conditions in Minnesota, North Dakota, South Dakota and Wisconsin. Progress reports on these investigations appear in 1924 *Proceedings*, A. S. T. M.; in *Concrete* for June, 1924, and October, 1925; in *Public Roads* for June, 1924, and October, 1925.

Effect of Sulphate Soils and Waters on Portland Cement and Concrete.—(University of Saskatchewan, Saskatoon, Can.)

These investigations include both laboratory and field studies and have been under way for several years. The tests are being carried out under the auspices of a research committee of the Engineering Institute of Canada with the financial support of the Research Council of Canada, Canada Cement Co., Canadian Pacific R. R. and the three prairie provinces of Canada. Reports on data so far available were published in *Industrial and Engineering Chemistry* for May and June 1925, and in the *Engineering Journal* for Feb., 1926.

SULPHUR IMPREGNATION:

Effect of Sulphur Impregnation on Concrete.—(Yale University, New Haven)

TEMPERATURE:

Effect of Temperature on Concrete.—(Illinois Department of Public Works, Springfield)

Studies on conductivity of heat, temperature gradient, stresses due to temperature, coefficient of expansion, etc. Special beams with methods of heating and cooling. Work also conducted on pavement slabs and special slabs.

Effect of Low Temperature on Concrete.—(University of Kentucky, Lexington)

A study of the effect of keeping concrete cylinders frozen for 14 days at a temperature of 8 deg. F. and then curing at about 70 deg. F. until tested at 28 days. Freezing as soon as mixed reduced strength 40 per cent; freezing after 24 hours reduced strength 37 per cent. See *Concrete Products,* v. 28, p. 56, March, 1925.

Effect of Temperature on Early Strength of Concrete.—(New Mexico State Highway Department, Las Cruces)

An investigation on the effect of temperature on the early strength of concrete of different mixtures and consistencies. The operation of mixing and curing specimens carried out under different conditions of atmospheric temperature. The range in temperature noted by means of a recording thermometer and the effect of different temperatures on the early strength of 6 x 12-in. cylinders studied.

TEST METHODS:

Sources of Variability in Compressive and Tensile Test Results.—(University of Colorado, Boulder)

Includes study of:

(a) Factors in the making of specimens.
  1. Effect of time between first mixing and placing in molds.
  2. General technique of making.
  3. Several cylinders vs. single cylinder batches.

(b) Storage.
  1. Fresh vs. still or stagnant water.
  2. Delayed storage.
  3. Removed before test.
  4. Intermittent wetting.

(c) Testing.
  1. End condition.
  2. Rate of applying load, etc.

See "Effect of Varied Curing Conditions Upon the Compressive Strength of Mortars and Concretes," by Herbert J. Gilkey, 1926 *Proceedings,* American Concrete Institute.

Methods of Capping Test Specimens.—(University of Kentucky, Lexington)

A study of methods of capping test specimens which included capping with cement, Beaver board, plaster of paris and sheet lead. Tests showed that a perfect neat-cement cap was superior to plaster of paris by 20 per cent, to a troweled surface by 40 per cent, to Beaver board by 45 per cent and to sheet lead by 50 per cent.

Effect of Type of Cap on Results of Compression Tests of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

This investigation includes a study of different capping materials of varying thicknesses for different end conditions of specimen.

WATERPROOFING AND PROTECTIVE TREATMENTS: (See also Sea Water)

Tests on the Efficiency of Penetrative Treatments for Waterproofing Concrete.—(University of Wisconsin, Madison)

Comparison of the leakage through concretes of various densities before and after treatment with various common penetrative surface treatments for waterproofing and determination of the weathering properties of such treatments. Preliminary tests made on 15 common types of surface treatments. There appears to be a wide difference in the efficiency of the various treatments tried. Work now being done on development of a more satisfactory type of apparatus in order to determine more definitely the relative efficiency of the different surface treatments.

YIELD:

Yield of Concrete.—(Illinois Department of Public Works and Buildings, Springfield)

Yield determinations made in laboratory using proportions and gradations of aggregates found in investigation of field proportions with the "Void Bulk Meter." Includes a study of the effect of variation in gradation of aggregates which meet the specifications on yield of concrete.

Investigations on Reinforced Concrete.

ARCHES:

Tests of Skew Arches.—(U. S. Bureau of Public Roads, Washington)

In order to measure experimentally the distribution and intensity of abutment reaction in skew arches, a series of tests has been conducted on concrete arches, one-fourth actual size, constructed on 30 deg., 45 deg., and 60 deg. skews. The arches were tested under a uniform load applied through 42 symmetrically placed loading points on the arch ring. Abutment reactions were measured by means of the soil pressure cells developed by the bureau, which were so placed as to take both the vertical and horizontal reactions at one end of the arch. (For progress report see *Public Roads*, Nov., 1925.)

Test of Skew Arch Models.—(Princeton University, Princeton, N. J.)

Tests to determine completely the abutment reactions for all load conditions using 4 hard rubber arch models made to 30 deg. and 60 deg. skew of same proportions as those tested by U. S. Bureau of Public Roads. Methods employed similar in principle to those described in January, 1925, *Proceedings* of the Am. Soc. of C. E. Special apparatus employed for deforming arches by foundation movements; measuring microscopes used to read arch deflections.

Investigation of Concrete and Reinforced-Concrete Arches.—(Special Committee on Concrete and Reinforced-Concrete Arches of the Am. Soc. of C. E.)

This committee is carrying out extensive studies on arches under construction and elaborate tests of laboratory models. Field work on bridges at Conneaut, Ohio, and Danville, Illinois, has been completed and the results are being studied. A number of other projects are under way or in contemplation. Extensive tests of concrete model arches have been conducted and are under way at the University of Illinois. The committee is planning to design a multiple-arch reinforced-concrete bridge for testing purposes.

BEAMS:

Investigation of Reinforced Concrete.—(U. S. Bureau of Standards, Washington)

(a) Shear tests of reinforced-concrete beams.

(b) Tests of reinforcing bars rolled from new billets and scrap steel.

Stresses in Web Reinforcement in Reinforced-Concrete Beams.—(University of Illinois, Urbana)

Numerous tests have been made in recent years to determine the stresses in web reinforcement in reinforced-concrete beams.

Stresses in Stirrups in Reinforced-Concrete Beams.—(Washington University, St. Louis)

A study of the value of present formulas for stirrup design. Stresses in stirrups determined by means of Berry strain gage using 2-in. gage length and comparison of observed stresses made with theoretical stresses. Relatively small beams used with loads at third points. Little or no stress observed in the stirrups under working loads. After concrete has failed in diagonal tension, stresses localize at stirrup which crosses the line of failure.

Construction Joints in Reinforced-Concrete Beams.—(University of South Dakota, Vermillion)

Comparison of the relative efficiencies of commonly used methods. Preliminary study made of tests on 100 beams. Vertical and 45-deg. joints at center and at ⅛ points of beam were tested using six different methods of bonding. Best type of joint is a combination of vertical plane on compression side and a 45-deg. plane on tension side; brushing with neat cement and using retempered concrete after scrubbing old face gave good results, the failure breaks not occurring at joints; use of 10 per cent of hydrated lime in joint mortar a failure.

Effect of Reinforcing Worn Bridge Floor Beams with Gunite.—(Engineering Experiment Station, Ohio State University, Columbus)

Tests to determine whether gunite will adhere sufficiently to steel to cause steel and concrete to act together. Worn steel beams thoroughly cleaned and gunite applied on mesh and reinforcing steel; beams tested in testing machine.

CHIMNEYS:

Tests of a Reinforced-Concrete Chimney.—(American Concrete Institute Committee S-1 on Reinforced-Concrete Chimneys)

These tests are being carried out on a 309-ft. reinforced-concrete chimney recently completed at Duluth, Minn., by the Universal Portland Cement Co. Among the factors studied are the following:

(1) Temperature drop through chimney walls at various heights.
(2) Measurement, by means of strain gages, of stresses in steel and concrete due to dead and wind load.
(3) Lateral movement of chimney due to wind.
(4) Wind velocities and pressures.

COLUMNS:

Tests of the Efficiency of Certain Types of Splices Used in Reinforced-Concrete Columns.—(University of Wisconsin, Madison)

Comparison of the strength and stiffness of columns reinforced with splices varying in length of lap with the similar properties of columns reinforced with unspliced rods. Includes study of the efficiency of pipe-sleeve splices in which the adjoining rods are butted. Thirty-six 8-in. x 5-ft. columns reinforced with 1 per cent spiral and with 2.8 and 5.8 per cent longitudinal steel were tested. Length of lap ranged from 2 in. to 40 diameters. Effect of tight and loose pipe sleeve and of sawn and sheared ends at the splices also investigated. Deformations in columns read over 10-in. gage lengths located at the splice and near the end of column. A splice with plain round bars lapped 40 diameters appears to be the most satisfactory of the types tested.

Investigation of Reinforced-Concrete Columns.—(University of Illinois, Urbana)

A general investigation which has been in process for several years. The tests are intended to bring out the effect of the amount and nature of the longitudinal and lateral reinforcement, the effect of age of concrete, the effect of length of column and of shrinkage, eccentric loading, splicing, and of a variety of other factors. Considerable data have been accumulated and gaps are being filled in.

Tests of Reinforced Concrete Columns.—(University of Pennsylvania, Philadelphia)

Comparison of strength of 12 x 12-in. x 15-ft. columns made under field conditions by the Turner Construction Co. from slag, gravel and broken stone aggregate. Measurements of deformations made with 20-in. Berry strain gages under continued loading approximating the allowable load. Marked differences in modulus of elasticity noted for gage lengths at top, middle and near bottom of columns; lowest values of modulus of elasticity observed at top of column.

PILES: (See also Sea Water)

Concrete Piling Investigation.—(Board of Harbor Commissioners, city of Los Angeles)

An extensive investigation covering laboratory tests of concrete and field tests of Raymond, Conzelman, duocrete and gunite concrete piles which have been suggested for use in connection with construction work of Los Angeles harbor. The tests included density and absorption tests on 6 x 6 x 12-in. concrete prisms; permeability tests on 7 x 12-in. concrete cylinders; compression, repeated load and static tests on 6 x 12-in. concrete cylinders; modulus of elasticity tests on 6 x 24-in. hexagonal concrete prisms; bond tests of both smooth and deformed square and round steel bars embedded in 6-in. concrete cubes; transverse tests on 14 x 14 in. x 14 ft. reinforced-concrete beams; driving tests of full-size concrete piles 40 ft. in length. Careful inspection made of concrete piling driven approximately 13 years ago in Los Angeles harbor. Complete details of investigation are given in report by W. R. Sadler, chief chemist, Los Angeles Harbor Department, Nov. 20, 1925.

PIPE:

Effect of Concrete Cradles on the Cracking Strength of Highway Culverts.—(Iowa State College, Ames)

A study of the effect of various types of plain and reinforced-concrete cradles on strength of concrete culvert pipe 24 to 84 in. in diameter. Field work completed. Data soon to be published.

SLABS:

Effect of Width of Reinforced-Concrete Slab on Unit Shearing Strength.—(University of North Dakota, Grand Forks)

Determination of the relation between the width of reinforced-concrete slab and the load which will cause failure in shear (or diagonal tension).

An Investigation of Steel I-Beams as Reinforcement for Concrete.—(McGill University, Montreal)

Investigation completed and in process of publication.

GENERAL:

Study of the Nature of the Resistance of Concrete to External Forces.—iversity of Illinois, Urbana)

his research involves tests of concrete restrained laterally by spirals and ted to external pressure in two directions.

PART II—SUGGESTED RESEARCHES ON CONCRETE MATERIALS, PLAIN AND REINFORCED CONCRETE.

CEMENT:

Development of a suitable test to replace the present normal consistency test of cement.

Improvement of method of compacting mortar cylinders.

Quicker and more uniform tests on portland cement.

Early strength portland cement and laboratory tests for the same.

The relative value of portland as compared to high alumina cement.

Studies of the strength relations, durability and long time tests of new special cements under various conditions.

AGGREGATES:

Relative value of the various grades of slag for coarse aggregate as compared with crushed stone or gravel. Also value of using prewet or saturated porous aggregate such as slag and cinders compared with the same dry aggregate. Methods of determining quality of slag aggregates.

Development of light-weight concrete for fire-proofing, steel bridge floors, roof slabs and for insulation. May be artificially made or made by specific treatment of natural light-weight aggregates to improve their strength.

Method for determination of shale in sands when the specific gravity of shales is about the same as that of other ingredients.

Development of satisfactory tests for aggregate.

Rational tests for coarse aggregate—especially crushed limestone. Present tests mean very little.

Ratio of dry rodded volumes of aggregates to the volume in the batchers.

Method for accurate control of the volume of fine aggregate on highway construction work.

Methods for predicting the volume of a mixture of two or more aggregates from the mechanical analyses of these aggregates.

Development of standard strength tests using quick-hardening cements like Lumnite, in which the quality of aggregates may be known within 24 hr. and probable performance with Portland cement predicted.

Use of Lumnite cement in making 1-day tests to determine the suitability of sand.

Method for accurate control of gradation of coarse aggregate on highway construction work.

Effect of variation in grading of coarse aggregate on strength and yield of concrete.

Fire resisting value of various aggregates.

Durability of concrete aggregates as shown by soundness and by freezing and thawing tests.

Relation between results of soundness tests on coarse aggregates and their durability in concrete.

Effect of flat pieces of gravel on the strength of concrete.

Effect of dust coating on aggregates on properties of resulting concrete.

Relationship of strength of aggregate to resulting strength of concrete.

Determination of the proportion of the mixing water which becomes a permanent part of the final concrete.

Method for accurate control of water content of concrete.

What is safe limit of alkali salts in water used for mixing concrete?

PLAIN CONCRETE:

Effect of sulphates on the deterioration of concrete.

Do surfaces subjected to multiple treatment with dilute sodium silicates show an improved resistance to sulphate soils?

Determination of value of protecting coatings for concrete in sea water.

Durability of high alumina cement concrete in sea water.

The efficiency of integral mixtures and of surface coatings in water-proofing concrete.

The abrasive resistance of concrete floor hardeners.

The value of impregnating finished concrete surfaces with silicate solutions for the prevention of wear.

Effect of high inorganic silt or clay, etc., particularly in the leaner mixtures, on strength, permeability and weathering resistance of concrete; the fine material to be considered as part of the fine aggregate in the given mixtures, not as an addition to the mixtures.

Does low uniformity in concrete contribute to cracking?

Methods of adding to the extensibility of concrete surfaces to prevent crazes and other cracks that diminish the life of structures; the first thing to do is to study the values of unit extension under various conditions.

Development of tests to determine durability of concrete and aggregates exposed to weather.

Causes of surface checking on portland cement concrete pavements.

Study of workability of concrete mixtures and development of an improved method for testing consistency.

Value of certain admixtures in increasing workability.

Economy of the use of admixtures to secure ease of placement and greater uniformity.

Effect of calcium chloride on different cements when employed as admixture for curing.

Effect of calcium chloride admixtures on reinforcing steel.

Determination of value of admixtures such as diatomaceous earth, trass, etc., in rendering concrete impervious to sea water.

Acceleration of hardening of concrete.

Comparative volumetric changes in concrete due to moisture, when different coarse aggregates are used, for example, dense limestone and igneous rocks, porous sandstones, gravel and blast furnace slag.

Accurate method for determining the cement content in hardened concrete.

Autogenous healing of concrete after being ruptured. This study to include all conditions.

Effect of age of concrete upon absorption.

The relation of the absorptive capacity of concrete to its density, strength, watertightness, and resistance to freezing.

Comparative strength of molded 6 x 12 and 8 x 16-in. cylinders and cores from concrete slabs.

Methods of surface treating artificial cut, cast, or faced stone to prevent rapid weathering, crazing, staining, etc.

Investigation of the comparative properties (strength and impermeability) of concretes made with lumnite and portland cement.

Study of effect of impact on different mixtures of concrete and different thicknesses of pavement slab.

Effect of leaching action of water on concrete; including pure water as well as water containing vegetable acids found in ground water.

Strength of concrete bricks, with regard to their properties and specifications.

Influence of silt and clay content in sand on scaling of concrete roads.

Relation of compressive strength of neat cement to strength of concrete.

Tensile strength of concrete.

Effect of petroleum products on concrete.

The elimination of shrinkage in concrete.

Specifications based upon actual tests for concreting in cold weather (a) 40 deg. F. to 32 deg. F.; (b) 20 deg. F. to 30 deg. F.; (c) below 20 deg. F.; (1) for concrete structures; (2) concrete pavements and bases. Should cover use of calcium chloride, heating water only, heating water and aggregates, methods of curing—temperatures and moisture conditions, methods of protecting, etc.

Effect of steam curing of concrete under different conditions.

Presentation of strength curves in simple form at various temperatures to assist in predicting future strengths.

The effect of temperature on strength of concrete at different ages for various mixtures and consistencies.

Further tests on curing of concrete pavements with calcium chloride.

How long or what set concrete should have before surface curing with calcium chloride.

A study of the effect of repeated freezing and thawing on concrete exposed to percolation by water, as in concrete tanks, arch dams, etc.

Effect of heat upon strength of concrete.

A study of weathering of concrete.

Variation in strength of concrete mixed by different types of mixers.

REINFORCED CONCRETE:

Effect of shrinkage on bond.

Values for bond between steel and Lumnite cement concrete.

Study of the strength of wedge-shaped or tampering beams, as used in brackets, footings, retaining walls, etc.

Investigation of the stresses developed in reinforced-concrete wing walls or other retaining walls of rapidly varying height.

Improving design for more continuous and smaller elements.

Method of protecting reinforcing steel against corrosion in plain and ornamental street lighting poles: coating materials, galvanizing, cement dip, etc.

Value of silicate solutions as a means of controlling corrosion of reinforcing rods.

Protection of steel from corrosion.

Combating corrosion of steel and concrete in relatively small members.

Strength of cinder concrete; effect of sulphur content of cinders upon reinforcement.

Actual stresses in concrete domes.

Torsional stresses in beams; for example, spandrel beams carrying eccentric wall loads.

Studies of size of tension cracks necessary to affect embedded steel.

Studies of required embedment of steel under various moisture and temperature conditions.

Exact action and effectiveness of the usual shear reinforcing.

Study of stresses in stirrups of reinforced-concrete beams.

Diagonal tension reinforcement in beams and girders.

The effect of age on the strength of spirally-reinforced columns.

Connections for reinforced rods in concrete compression members.

Investigation as to increased cost of structures due to unreasonable demands in fire protection such as excessive thickness and use of special materials compared with probable annual losses due to fire damage on reinforced-concrete structures.

The strength and other properties of reinforced-concrete structural units built up from individual concrete blocks without the use of forms.

Relative merits of structural, intermediate, hard and rail steel reinforcement for reinforced-concrete pavements.

APPENDIX B.

Recent Papers and Reports on Researches in the Field of Concrete.

Note: Papers published in the *Proceedings* of the American Concrete Institute are not included in the following list. The brief notes which follow several of the references reflect the views of the authors of the papers and do not necessarily represent the opinion of the committee.

Part I—Papers and Reports on Researches Carried Out in the United States and Canada.

CEMENT:

Properties and Use of Aluminate Cement, by R. L. Morrison;
Concrete, v. 26, p. 128, April, 1925.
Pit and Quarry, v. 9, p. 71, March 1, 1925.
Michigan Roads and Pavements, v. 22, p. 15, Feb. 26, 1925.
Highway Engineering and Contracting, v. 12, p. 41, April, 1925.
Engineering and Contracting, v. 63, p. 187, May 6, 1925.
Chemical analysis and physical tests on mortar concrete for different mixes, curing conditions and consistencies at ages up to 83 days. Effect of addition of portland cement on setting time shown.

Set of Lumnite Cement, by S. L. Meyers;
Concrete, v. 26, p. 85, March, 1925.
Rock Products, v. 28, p. 49, Feb. 7, 1925.
Chemical analysis and physical tests of Lumnite cement. Effect of temperature on initial set; effect of percentage of water used in gauging and effect of fineness of grinding on set. Effect of numerous acids, chlorines, bases, inorganic salts and inorganic compounds on initial and final set determined. Comparison is made of various conditions and chemicals on set of Lumnite and portland cements.

Temperatures in High-Alumina Cement and Methods of Curing, by H. S. Mattimore;
Proceedings A. S. T. M., v. 25, 1925.
Engineering News-Record, v. 95, p. 64, July 9, 1925.
High-alumina cement has compressive and transverse strengths at 24 hr. greater than those of portland-cement concrete at 28 days. Wet burlap curing increases temperature during hydration and produces defective surface on high-alumina cement concrete. Ponding or sprinkling of high-alumina cement concrete at early stages produces scaled surface. Air curing of high-alumina cement concrete reduces high temperature during hydration.

New Testing Devices Developed;
Public Roads, v. 6, p. 68, May, 1925.
Device designed to register intensity of pressure in molding of cement mortar briquets. Briquets molded on scale platform.

Requirements of Cement for Modern Highway Construction, by A. T. Goldbeck;
Proceedings A. S. T. M., v. 25, 1925.
Engineering News-Record, v. 95, p. 64, July 9, 1925.
Service requirements, fineness, soundness test, tensile strength and time of set considered.

Disintegration of Portland Cement in Sulphate Waters, by Thorvaldson, Harris, and Wolochow;
Industrial and Engineering Chemistry, v. 17, p. 467, May, 1925.
Solutions of sodium sulphate, magnesium sulphate, and mixtures of these shaken with set portland cement and then analyzed. Procedure of tests and action of sulphates and chlorides on set cement discussed.

Action of Sodium and Magnesium Sulphates on Constituents of Portland Cement, by G. R. Shelton;
Industrial and Engineering Chemistry, v. 17, p. 589, June, 1925.

Describes preparation in pure state of major substances present in normal portland cement clinker, tricalcium aluminate, beta dicalcium silicate, and tricalcium silicate and effects of these constituents by solutions of sodium and magnesium sulphates of various concentrations. Solids only were investigated and changes brought about by the solutions were noted with aid of petrographical microscope.

Effect of Fine Grinding of Portland Cement on Strength of Concrete; Tech. News Bull. 99, U. S. Bureau of Standards, July, 1925.

Tests up to 10 yr. on strength of 1 : 2 : 4 and 1 : 3 : 6 concrete made with cements of various finenesses. Results indicate that fineness of cement increases strength of concrete; all cements do not give same increase in strength with same increase in fineness; effect of fineness of cement on strength of concrete diminishes with age; and 1 : 2 : 4 mix showed better increases in strength with same increase in fineness than 1 : 3 : 6 mix.

Long-Time Tests of Unsound Cement;
Tech. News Bull. 96, U. S. Bureau of Standards, April 10, 1925.

Compressive strength tests on 1 : ½ : 3 concrete and tensile strength of 1 : 3 standard sand briquets under different storage conditions at 6 years.

Photometric Method for Magnesia in Portland Cement, by W. E. Hacke; *Concrete,* v. 27, p. 1 (CMS), July, 1925.

Comparison of photometric and gravimetric methods.

AGGREGATE:

Grading of Aggregates, by R. W. Crum;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 117.

Results of Feret, Fuller, Abrams, Edwards, and Talbot on design of concrete mixtures reviewed.

Suggests One-Day Strength Tests for Concrete Aggregate, by Slack and Boyd;
Engineering News-Record, v. 94, p. 1014, June 18, 1925.

Quick-hardening alumina cement gives results comparable to same sand tested with slower hardening portlands. Tension tests of briquets and compression tests of mortar and concrete of alumina cement at 24 hr. and portland cement at 28 days with different sands. Tests made by Georgia State Highway Department.

Soundness Tests of Coarse Aggregates, by M. O. Withey;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 108.

Four types of crystallization tests used for determining soundness of stone and clay products. Discussed (a) Freezing and thawing tests, (b) Sodium sulphate test, (c) Sodium Chloride test, (d) Alkali test. Method of making tests, details and results of soundness tests on various aggregates described.

PLAIN CONCRETE:

Absorption of Concrete Water as Affected by Aggregates—Its Ultimate Effect in Expansion of Road Slabs, by F. C. Lang;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 129.

Discusses tests at Structural Materials Research Laboratory, U. S. Dept. of Agriculture, University of Illinois, and Minnesota Highway Dept.

New Testing Devices Developed;
Public Roads, v. 6, p. 68, May, 1925.

Field apparatus for testing consistency of concrete. Cone which holds about 75 lb. of concrete with a movable bottom is placed above a scale. When the bottom is removed, the fresh concrete flows on a round scale platform 15 in. in diameter. Very wet or very dry concrete flows off the plate. For machine-finished concrete the proper consistency will give the greatest weight of concrete retained on the plate. See also "A New Test for Consistency of Paving Concrete," by Jackson and Werner, Proc. A. S. T. M., v. 25, 1925.

Studies of Curing Concrete in a Semi-Arid Climate, by Gonnerman and McKesson;
Bull. 15—Structural Mat. Research Lab., Lewis Institute, Chicago, 1925.

Flexural tests of plain concrete beams at ages of 3 to 90 days in study of relative efficiency of 5 methods of curing. Curing methods included covering with wet earth and asphaltic paper, surface applications of flake calcium chloride and of sodium silicate, and air curing. Surface hardness of cured concrete was measured by means of a ball indentation test. Tests

made at Sacramento in co-operation with California Highway Commission.

On Calcium Chloride for Curing, by C. L. McKesson;
Western Highways Builder, v. 7, p. 18, Feb., 1925.
Calif. Highways, v. 2, p. 5, Feb., 1925.
Sand and Gravel Bull., v. 6, p. 6, April 5, 1925.
Tabulation shows efficiency of calcium chloride in curing concrete as determined by compression tests on 6 x 6 x 12-in. concrete prisms and 4½-in. cores from pavement. $CaCl_2$ is 80 to 90 per cent efficient and only recommended where water is scarce.

Tests on Concrete Drain Tile;
Engineering and Contracting, v. 64, p. 615, Sept. 9, 1925.
Describes tests by U. S. Dept. of Agriculture, Minn. State Dept. of Drainage and Waters and Univ. of Minn. on concrete drain tile exposed to alkali soils and waters in Minnesota and Medicine Lake, S. D., three thousand 2 x 4-in. cylinders made from 125 different types of concrete to determine which concretes were most resistant to alkali attack.

Report on Field Tests on Concrete Used in Construction Work, by Slater and Walker;
Proc. Am. Soc. C. E., v. 51, 1925.

Results of Field Tests of Bridge Concrete, by H. D. Miller;
Calif. Highways, v. 2, p. 7, April, 1925.
Engineering News-Record, v. 95, p. 182, July 30, 1925.
Concrete, v. 27, p. 49, Aug., 1925.
10- and 28-day strength tests now standard practice by California Highway Commission.

Investigation of Effects of Fire Exposure Upon Hollow Concrete Building Units. Conducted by Underwriter's Laboratories for American Concrete Institute, Concrete Products Association and Portland Cement Association;
Retardant Report 1555 of the Underwriter's Laboratories, Inc., May, 1924.

Fire Resistance of Concrete Columns, by Hull and Ingberg;
Tech. Paper 272, U. S. Bureau of Standards, p. 677, 1925.
Fire and Water Engineering, 1925.
Canadian Engineer, v. 49, p. 213, Aug. 11, 1925.
Fire tests on 62 columns under load with wide range of concrete aggregates; thickness of covering 1½ to 2½ in. Quartz, chert or granite induced spalling and cracking, while limestone or calcareous gravel showed little visible effects.

Frozen Concrete, by D. V. Terrell;
Scraper, v. 5, p. 14, March 13, 1925.
Am. Road Builder, July, 1924.
Concrete Products, v. 28, p. 56, March, 1925.
Compressive strength of 1:2:4 and 1:2:3 concrete frozen and allowed to remain frozen for 14 days after setting 24 hr. and for 30 min. in laboratory. Specimens broken at 27 days after freezing period.
Conclusions were:
(1) Concrete having attained its final set before freezing added to its strength only 41.7 per cent.
(2) Freezing of 1:2:4 concrete 14 days reduced its strength 35.5 per cent.
(3) Freezing 1:2:3 concrete 14 days reduced its strength 35.5 per cent.
(4) Richer mix not affected as much as leaner.

Direct Measurement of Poisson's Ratio for Concrete, by A. N. Johnson;
Proc. A.S.T.M., v. 24, part 2, p. 1024, 1924.
Describes mirror compressometer used in measuring deformations in concrete under vertical pressure.

Making Uniform Concrete by Inundating Sand, by R. L. Bertin;
Engineering News-Record, v. 94, p. 775, May 7, 1925.
Details and use of practical device to insure same amount of water regardless of moisture in sand.

Deterioration of Sheet Lead in Contact with Portland Cement Mortar and Concrete;
Concrete, v. 26, p. 33, January, 1925.
Tests at U. S. Bureau of Standards on 6-pcs. of cable imbedded in stiff portland cement mortar; 3 specimens coated with plain mortar and 3 specimens set in mortar containing 2 per cent of sodium silicate by weight of cement, stored as follows: air of laboratory, wetted occasionally; immersed in water to within 1 in. of the top of mortar; moist air of a damp closet. Recommendations for use of sheet lead for shower baths and similar installations given.

Researches in Concrete, by W. K. Hatt;
Bull. 24, Purdue Univ. Eng. Exp. Sta., Lafayette, Ind., 1925.
Survey of researches into nature and behavior of portland cement concrete. Presents sequence and extent of research activity, reviews status of some of the important fundamental researches in unsettled fields and gives a bibliography of researches in selected fields. Results are presented rather than conclusions.

Lowering Cost of Concrete in Pacific Northwest, by I. L. Collier;
Bull. 31, Univ. of Washington, Eng. Exp. Sta., 1925.
Compression tests on 600 6 x 12-in. cylinders in which the amounts of sand, gravel, cement and water were varied.

Status of Motor Truck Impact Test of Bureau of Public Roads, by C. A. Hogentogler;
Public Roads, v. 5, p. 11, Nov., 1924.
Engineering and Contracting, v. 63, p. 54, Jan. 7, 1925.

Study of Impact in Its Relation to Pavement Design, by G. W. Hutchinson;
Engineering News-Record, v. 94, p. 439, March 12, 1925.
Resistance to impact affected by thickness of concrete slab, coarseness of aggregate and cement content.

Impact and Static Loads of Road Slabs, by E. B. Smith;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 42.
Resistance of road slab depends on supporting value of subgrade; reinforcing steel in concrete slabs adds to resistance of slab to impact. Impact resistance of plain concrete slab greater on plain subgrade. No additional strength by addition of 2-in. bituminous top for either impact or static loads.

Movement of Concrete Due to Moisture, by A. H. White;
Concrete, v. 26, p. 41, January, 1925.
Tests on 11 specimens of neat cement, and 1-3 mortar from different brands of cement. Tests included studies to determine effect of ageing, effect of additions of finely-ground slag and hydrated lime, and tests also made of 31 expansion bars from 6 different types of synthetic cements burned in a laboratory kiln.

Effect of Moisture on Concrete, by W. K. Hatt;
Public Roads, v. 6, p. 14, March, 1925.
Proc. Am. Soc. C. E., v. 51, p. 757, May, 1925.
National Sand and Gravel Bulletin, v. 6, p. 7, May 15, 1925.
Investigations conducted by Eng. Exp. Sta. at Purdue University in co-operation with U. S. Bureau of Public Roads, for purpose of measuring maximum warping and surface deformation of concrete road slab resulting from non-uniform distribution of moisture as basis for estimating possible initial stresses; effect of moisture changes on modulus of rupture and compressive strength of concrete and volume changes in concrete beams due to exposure and saturation studied.

Fundamental Cause of Disintegration of Concrete, by A. H. White;
Concrete, v. 26, p. 157, 1925.
Disintegration caused by volume changes due to changes in moisture content of concrete. Frost action and corrosion of reinforcing steel is secondary. Expansion of neat cement bars stored under water after 24 hr. for 7 days to 20 yr.

Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads, by A. T. Goldbeck;
Public Roads, v. 6, p. 117, Aug., 1925.

Theory of Stresses in Road Slabs, by H. M. Westergaard;
Proc. 4th Meeting Highway Research Board, 1925, p. 60.
Discusses corner break, deflections and stresses caused by wheel load at considerable distance from edge of slab, effect of changes of temperature, blow-up of road slab due to expansion, etc.

Traffic Stresses Produced in Concrete Roads, by A. T. Goldbeck;
Proc. 4th Meeting Highway Research Board, 1925, p. 62.
Highest tensile stresses exist along edges of slab of uniform thickness. At unsupported transverse joint, comparatively high tension can exist either on top or bottom of slab when wheel load passes over joint. Stress in interior portion of plain slab at some distance away from transverse joint is considerably less than stress along outer edge.

Design of Concrete Pavements, by A. T. Goldbeck;
Good Roads, v. 68, p. 175, August, 1925.
Subgrade tests, treatment, etc., expansion and contraction of concrete due to moisture condition, compressive strength, modulus of rupture, modulus of elasticity, fatigue and tensile strength of concrete.

Recent Conclusions in Highway Research, by A. T. Goldbeck;
Assoc. of State Highway Officials, 1924.
Public Roads, v. 5, p. 9, Jan., 1925.
Engineering and Contracting, v. 63, p. 457, March, 1925.
National Sand and Gravel Bull., v. 6, p. 19, Feb., 1925.
Am. Highways, v. 4, p. 16, Jan., 1925.
Concrete, v. 26, p. 91, March, 1925.
Discusses subgrade, improvement of bed subgrade experiments in connection with road design, stress measurements in concrete pavements, impact experiments on bridges, earth pressure on bridge abutments or retaining walls, investigations of characteristics of aggregates suitable for concrete pavements, etc.

Recent Developments in Construction of Cement-Concrete Roads, by H. E. Breed;
Canadian Engineer, v. 49, p. 366, Sept. 29, 1925.
Beneficial effect of reinforcing, research of service conditions, adequate treatment of sub-grade, use of tar paper, rapid setting cements, split slab construction, etc.

Reinforcing and Subgrade Factors in Pavement Design, by J. T. Pauls;
Public Roads, Oct., 1924, p. 1.
Concrete, v. 26, p. 40, January, 1925.
Conclusions of observations on Columbia Pike, and experimental road constructed by the U. S. Bureau of Public Roads near Arlington, Va., after 2½ yr. service under traffic.

Stress Measurements in Concrete Pavements, by A. T. Goldbeck;
Public Roads, v. 5, p. 11, Jan., 1925.
Describes procedure and apparatus for making stress measurements in concrete pavements. Conclusions of tests.

Report of Investigation on Economic Value of Reinforcement in Concrete Roads, by C. A. Hogentogler;
Proc. 5th Annual Meeting Highway Research Board, 1926.
Summary of conclusions published by Highway Research Board.

Report of Tests on Concrete Block Made in Pittsburgh, Pa.;
Concrete Products, v. 28, p. 37, June, 1925.
Effect of kind and grading of aggregate, mix, time of mixing and curing conditions on strength and absorption of concrete block. Results indicate sands of coarse grading produced greater strength; certain kinds of fine material in sand increases workability, thereby increasing strength; concrete for tamped products should be mixed for at least 2 min. after adding water.

Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete, by H. F. Gonnerman;
Proc. A. S. T. M., v. 25, 1925.
Bull. 16, Structural Materials Research Lab., Lewis Institution, Chicago.
Compression tests at 7 days to 1 year on 1,755 cylinders, cubes and prisms in study of influence of size and shape of specimen on compressive strength. Tests covered wide range of mixes, consistencies, sizes and gradings of aggregate.

"Analytical Properties of Set and Hardened Mortars," by E. E. Butterfield;
Proc. A. S. T. M., v. 25, 1925.

Prediction of 28-day Breaking Strengths of Mortars from Their 7-day Results, by Gowen, Leavitt and Evans;
Maine Technology Exp. Sta., Bull. No. 10, 1925.

"The Significance of the Common Test Methods for Determining the Strength of Mortars," by Gowen and Leavitt;
Proc. A. S. T. M., v. 25, 1925.

Comparison of Strength of Concrete in Tension and Compression, by N. M. Finkbinder;
Public Roads, v. 5, p. 14, Jan., 1925.
Results of tension tests on 6 x 6-in. tension specimens and 6 x 12-in. compression cylinders indicate that compressive strength is about 11 times tensile strength.

Decreased Strength of Concrete When Wet, by H. S. Mattimore;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 130.
Reviews tests of Van Ornum, Woodward and Young and Lagaard. Tabulation of strength tests by Minnesota Highway Department.

Water Ratio Specification for Concrete, by McMillan and Walker;
Forthcoming Proc. Am. Soc. C. E., May, 1926.

Significance of Talbot-Jones Rattler as Test for Concrete in Road Slabs, by H. H. Scofield;
Proc. 4th Annual Meeting Highway Research Board, 1925, p. 127.
Reviews work of Abrams, Mattimore, Jackson, Goldbeck and others.

REINFORCED CONCRETE:

Concrete Beams;
Tech. News Bull. 98 U. S. Bureau of Standards, June, 1925.
Results of resistance to shearing stresses of 172 reinforced-concrete beams of I-section of web thickness from 2 to 12 in. Depth of beam 36 in. on span of 9½ ft. or 52 in. on span of 16 ft. Strength of concrete varied from 2,100 to about 5,400 per sq. in.

Studies of Bond Between Concrete and Steel, by D. A. Abrams;
Proc. A. S. T. M., v. 25, Part II, 1925.
Bull. 17, Structural Materials Research Lab., Lewis Institute, Chicago.
735 pull-out tests on 1-in. plain round steel bars embedded in 8 x 8-in. concrete cylinders and 735 parallel compression tests on 6 x 12-in. cylinders at ages of 7 days to 1 year. Concrete covered wide range in size and grading of aggregate, quantity of mixing water and cement. Hydrated lime and crude oil used as admixtures in a few tests.

PART II—PAPERS AND REPORTS ON RESEARCHES CARRIED OUT IN FOREIGN COUNTRIES.

CEMENT:

Cement Research in Germany in 1923 and 1924, by H. Nitzsche;
Tonind. Ztg., No. 22, 1924.

Latest Cement Research, by K. Endell;
Zementverlag, Charlottenburg, 1925.

Report of Laboratory of Association of German Portland Cement Manufacturers, by G. Haegermann;
Zement No. 26 and 27, July 2 and 9, 1925.
Summary of investigations carried out through the year comprising tests of fused and alumina cements, effect of temperature, water glass admixtures, etc.

Use of X-Ray in Cement Research, by R. Nacken;
Zement No. 19 and 20, May 14 and 21, 1925.
Presented at annual convention of German Portland Cement Mfr.

Contribution to Petrography of Modern Portland Cement Clinker, by C. Biehl;
Zement, v. 14, p. 379, 397, 1925.
By microscopical investigation of clinker it is possible to draw conclusions as to firing, sintering, kind of kiln, cooling, influence of fluxes, chemical composition, and quality of cement made of that clinker. Best cements obtained by suddenly-cooled clinkers, where a complete crystallization of minerals is prevented and a storage of energy in the glass is reached which becomes effective in cooling.

Processes Involved in Calcination of Raw Mix, by W. Dyckerhoff;
Zement, No. 10, p. 200, March 12, 1925.

Investigations of Alumina Cement and Concrete, 1924, by H. Kreuger;
Report No. 2, Royal Tech. School, Stockholm.

Effect of Water and Salt Solutions of Alumina Cement, by Haegermann and Hart;
Zement No. 10, March 12, 1925.

Examinations of Cements High in Alumina, by Lindeman and Hassel;
Tids. Kemi. Bergvaesen, v. 4, p. 149, 193, 1924.
Mixtures of pure Al_2O_3, SiO_2 and $CaCo_3$ fused in electric furnace cooler, and ground to 15 per cent residue on 4,900-mesh sieve. Time of set and tensile and compressive strength of tests of normal sand mortars at 1 and 7 days. Effect of various composition of products ranging from 3-22 per cent SiO_2, 37-63 per cent $A\text{-}_2O_3$, 25-49 per cent CaO on hydraulic properties, setting, hardening and strength.

Setting of Cement, by H. Malgrain;
Rev. Mat. de Const. et Trav. Pub., v. 186, p. 57, v. 187, p. 91, 1925.
Setting of cement considered as being due to hydration of each of its constituents and is expressed as a number of first order reactions. Variations of duration of set function of temperature. Setting process divided into 3 stages. Tensile strength tests on cement show maximum strength at 3 mo. Lime liberated during setting. Amount of water combined with cement and eliminated by evaporation calculated.

Experiments on Setting of Cement, by J. Cartiaux;
Rev. Mat. de Const. et Trav. Pub., v. 184, p. 1, 1925.
Following influences studied: Free lime content, soluble salts, influence of air, influence of CO_2, loss of weight.
Ore Cement and Blast-Furnace Slag Cement in Sea Water, by C. Prüssing;
Zement, v. 13, p. 319, 1924.
After 20-yr. immersion in sea water ore-cement pats were sound, while pats of blast-furnace slag cement and iron portland cement developed networks of fine cracks during 10-yr. immersions. Ore cement is most resistant building material to action of sea water over long periods of time.
Magnesia Portland Cement, by K. Balthasar;
Tonind. Zrg., No. 19, Mar. 7, 1925.
Rock Products, v. 28, No. 10, p. 50, 1925.
Satisfactory portland cement containing 9 per cent mgO manufactured in vertical kiln by heating at clinkering temperature longer than usual for ordinary portland cement.
Effect of Storage on Slag Cement, by R. Grün;
Tonind. Ztg., No. 1, 1925.
Comparison of data on effect of storage of slag cement with that of portland cement.
Behavior of Alkalies in Cement During Water Storage of Cement and Cement Mortar Bodies, by V. Rodt;
Zement, v. 13, p. 470, 1924.
Cement and mortar briquets stored in tap water and lime sugar solutions; sugar delays set and accounts for increased alkali liberated from cement when stored in this solution.
Rapid Hardening Slag Cement, by Prevost;
Concrete, v. 26, p. 172, 1925.
Cement consisting of finely-ground mixture of 80 per cent granulated blast furnace slag and 20 per cent hydrated lime begins to set in 4 hr. and gives concrete of higher strengths than portland cement at ages up to 3 mo.
Rapid Hardening Ferrocrete Portland Cement, by O. Faber;
Pamphlet by Cement Marketing Co., Ltd. (London), 1925.
Abstract, Concrete and Const. Eng., v. 19, p. 747, Dec. 1924 and v. 20, p. 5, Jan., 1925.
Abstract, *Concrete,* v. 26, p. 130, April, 1925.
Rock Products, v. 28, p. 63, Aug. 8, 1925.
Practical strength tests of ferrocrete. Load and deflection tests on beams of ordinary and rapid hardening portland cement at 2 days to 3 mo.
Early Strength Tests, by C. Prüssing;
Proc. German Portland Cement Mfr., p. 100, 1924.
High Strength Cements, by O. Graf;
Zeits, Ver. Deutscher Ing., No. 33, Aug. 16, 1924.
High Strength Cements, by Gehler;
Proc. German Portland Cement Mfr., p. 35, 1924.
Tests at Dresden laboratory showing strength relations of ordinary portland, high-strength portland and special cements.
Compressive Strength of Cement Mortar, Concrete, Reinforced Concrete, Masonry, by O. Graf;
Konrad Witwer, Stuttgart, 1921.
Use of High Strength and Ordinary Portland Cement for Plain and Reinforced Concrete, by E. Probst;
Zement, March 12, 1925.
Relative merits of special high-strength cement and portland cement when used in plain and reinforced concrete.

PLAIN CONCRETE:

Agencies Affecting Concrete, by Dr. A. Kleinlogel;
Published by Ernst & Son, Berlin, 1925.
Chemical, mechanical and other agencies such as air, water, acids, bases, oils, vapors, soils, storage, etc., affecting cement, mortar, concrete and reinforced concrete and methods for reducing or eliminating their effect.
Effect of Powdered Admixtures in Cement, by R. Feret;
Rev. Mat. de Const. et Trav. Pub., No. 177-192, 1922-1925.
Calcium Chloride as an Agent for Protecting Cement and Concrete Work from Destruction by Frost, by A. Troche;
Zement, v. 13, p. 266, 1924.
Ten to 15 per cent anhydrous calcium chloride to 1:1 sand-cement mixes lowers freezing point 5 and 10 deg. respectively. Mixes do not set for at least 1 hour. Spots on mortars are not true efflorescence. Mortars containing calcium chloride do not show

lower crushing strength after storage in dry air as when stored in moist air.

Use of Calcium Chloride in Cement and Concrete as Protection Against Cold, by R. V. Frost;
Published by State Testing Laboratory, Stockholm, 1924.

Effect of Magnesium Sulphate on Mortar and Concrete, by L. Zimmermann;
Proc. German Portland Cement Mfr., p. 195, 1924.
Zement No. 16 to 19, 1924.

Action of Sugar on Properties of Portland Cement Mortars, by C. Tsountas;
Rev. Mat. de Const. et Trav. Pub., v. 182, p. 281, 1924.
Addition of small quantities of sugar to portland cement mortars decreases time of set and lowers strength of mortar. Assumes that hydrated lime set free during decomposition of tricalcium silicate, on addition of water combines with sugar to form saccharates of lime.

Tests to Determine Resistance of Concrete Specimens With and Without Trass, by O. Graf;
Deutscher Ausschuss für Eisenbeton, No. 43, Ernst & Son, Berlin, 1920.

Can Trass be Replaced by Other Admixtures? by H. Burchartz;
Zement No. 15 and 16, Apr. 16 and 23, 1925.

Questions Concerning Concrete in Sea Water, by O. Gassner;
Zement, No. 21 to 25, 1924.

Tests on Behavior of Mortar and Concrete in Bogs, by M. Gary;
Deutscher Ausschuss für Eisenbeton, No. 49, Ernst & Son, Berlin, 1922.

Investigations to Determine Causes of Deterioration in Structures Exposed to Water, by O. Graf;
Beton u. Eisen, No. 4, Eisen, No. 4, Feb. 20, 1925.

Experimental Investigations on Coefficient of Elasticity of Concrete, by G. Magnel;
Rev. Univ. des Mines, v. 67, p. 38, Jan., 1924.
Resistance increased with concrete between 3 mo. and 1 yr. and during same time coefficient of elasticity remains practically constant; permanent deformation under same load less at 1 year than at 5 mo.

Modulus of Elasticity of Concrete in Compression, by W. D. Womersley;
Concrete and Constr. Eng., v. 19, p. 553, Sept., 1924.
Tests on cylinders 2 ft. in diameter and 4 ft. long; 4-in. cubes; 3 in. diameter tension specimens of 1 : 2 : 4 concrete at 90 days.

Modulus of Elasticity of "Spun" Concrete in Tension;
Concrete & Constr. Eng., v. 19, p. 709, Nov., 1924.
Tests on Concrete poles 6 ft. long, 4 in. internal diameter and 6 in. external diameter. Transverse, compression and tensile tests made and results of tests tabulated.

Investigation of Relation of Compressive Strength of Cement to Concrete Strength, by H. Kreuger;
Report No. 1, Royal Tech. School, Stockholm.
Zement, No. 47, 48, 49 and 50, Nov. 27 to Dec. 18, 1924.

Tamped vs. Poured Concrete, A Study of Density, by R. Otzen;
Bauingenieur, No. 16, 1923.

Effect of Sand and Water Content on Consistency and Strength of Concrete, by A. Hummel;
Bauingenieur, v. 5, p. 817, Dec. 25, 1924.
Tests at Karlsruhe Technical High School on 1 : 6 concrete with increasing sand content but same water content and how to vary addition of water to maintain equal plastic consistency.

Effect of Grading and Silt Content of Sand on Concrete Strength, by H. Nitzsche;
Beton u. Eisen, No. 15, p. 195, 123.

Strength of Concrete for Variable Sand Content of Aggregate in Dry, Moist and Fluid Concrete, by M. Gary;
Deutscher Ausschuss für Eisenbeton, No. 51, Ernst & Son, Berlin, 1922.

Composition of Mortar in Concrete, by O. Graf;
Published by Julius Springer, 1923.

Hair Cracks on Concrete: Their Causes and Remedy;
Ferro Concrete, v. 16, p. 96, Nov., 1924.
Theories as to cause in concrete and stucco surfaces.
Recommends proper curing and elimination as far as possible of rich surface film. This elimination may be secured by better selection of aggregates, less trowelling, then wherever possible, rubbing, trowelling or washing surface.

Investigations and Tests of Permeability of Mortar and Concrete, by O. Graf;
Bauingenieur, No. 8, 1923.

Testing Permeability to Water of Mortars, Brick Stone, by F. Anstett;
Rev. Mat. de Const. et Trav. Pub., v. 183, p. 309, 1924.
Tissier apparatus for this purpose described.

Report on Quantities of Materials for Concrete Mixtures, by H. Kreuger; Report No. 3, Royal Tech. High School, Mat. Testing Lab., Stockholm, 1924.

Investigations to Determine More Rational Proportioning of Hydraulic mortar and Concrete, by R. Feret; Rev. l'Ingenieur, Sept., 1923.

Rational Proportioning of Concrete and Predetermination of Compressive Strength in the Field, by O. Graf; Gesundheits-Ingenieur, No. 26, 1923.

Volume Changes, Strength and Watertightness of Concrete Made from Portland Cement and High-Strength Alumina Cement, by A. Hummel; Bauingenieur, No. 5, 1924.

Further Investigations of Volume Changes of Concrete During Setting, by O. Graf; Beton, u. Eisen, p. 172, 1922.

Temperature Variations and Temperature Movements in Stone and Concrete Bridges, by F. Vogt; Published by Ernst & Son, Berlin, 1925.

REINFORCED CONCRETE:

Reinforced-Concrete Beams in Bending and Shear, by O. Faber; Published by Concrete Publications, Ltd., London, 1925.

Tests of Restrained Reinforced-Concrete Beams, by Bach and Graf; Deutscher Ausschuss für Eisenbeton, No. 45, Ernst & Son, Berlin, 1920.

Shearing Strength of Reinforced-Concrete Beams in Flexure, by O. Hausen; Beton u. Eisen, No. 18, Sept. 20, 1924.

Formulas and Computations. Highly mathematical discussion.

Tests of Reinforced-Concrete Beams to Determine Resistance of Various Reinforcement Against Shearing Forces, by Bach and Graf; Deutscher Ausschuss für Eisenbeton, No. 48, Ernst & Son, Berlin, 1921.

Torsion Tests to Determine Shearing Resistance of Reinforced Concrete, by Graf and Mörsch; Published by Springer, Berlin, 1922.

Steel in Concrete with Slag Cement, by M. Gary;

Tests on Bond Resistance of Galvanized Steel in Concrete, by F. Schmeer; Deutscher Ausschuss für Eisenbeton, No. 47, Ernst & Son, Berlin, 1920.

Tests of Bond Resistance in Reinforced-Concrete Beams, by W. A. Slater; *Engineering News-Record*, v. 94, p. 1050, June 25, 1925.

Experiments by Bach and Graf give valuable information for design of reinforcing bars. Johnson, diamond, lug, cup, round and "wave" bars tested. Load applied at ¼ points of 11.8-in. square beams, 7 ft. 1.04-in. long on span of 6 ft. 7.8-in. 1 : 2 : 4 concrete used.

Tests of Reinforced-Concrete Slabs Supported on Two Sides Under Concentrated Loading, by C. Bach and O. Graf; Deutscher Ausschuss für Eisenbeton, Ernst & Son, Berlin, Part 1, No. 44, 1920; Part 2, No. 52, 1923.

Stress Measurements in Flat Slabs, by Probst and Butzer; Bauingenieur, No. 6, 1925.

Wind and Heat Stresses in Design of Tall Reinforced-Concrete Chimneys, by K. Doring; Published by Ernst & Son, Berlin, 1925.

Review of methods of design in view of original tests.

Testing of Beams and Cubes in Control Tests, by W. Petry; Deutscher Ausschuss für Eisenbeton, No. 50, Ernst & Son, Berlin, 1922.

Load and Fire Test of Reinforced-Concrete Warehouse in Wetzlar, by M. Gary; Deutscher Ausschuss für Eisenbeton, No. 46, Ernst & Son, Berlin, 1920.

Tests with Slab Beams to Determine Influence of Repeated Load, Weather and Smoke Gases, by Amos; Deutscher Ausschuss für Eisenbeton, Ernst & Son, Berlin, Part 1, No. 53, 1924; Part 2, No. 54, 1925.

Investigation of Effect of Repeated Loading on Elasticity and Strength of Plain and Reinforced Concrete, by E. Probst; Reprint, Spec. Pub., Centennial Celebration Tech. Hochschule, Karlsruhe, 1925.

New Official Specifications for Reinforced Concrete; Published by Ernst & Son, Berlin, 1925.

Including (A) Reinforced-Concrete Construction, (B) Tile and Joist Floors, (C) Plain Concrete Construction, (D) Field Tests.

AMERICAN CONCRETE INSTITUTE
BUSINESS REPORTS

Annual Report of the Board of Direction to the Members of the American Concrete Institute—April 1, 1926.

This volume 22 of the Preceedings is the best report of the technical work of the organization. That this work is having a constantly increasing appreciation is evidenced in the growth of membership:

Feb. 1, 1920	............Membership............	428
Feb. 1, 1921	............Membership............	627
Feb. 1, 1922	............Membership............	806
Feb. 1, 1923	............Membership............	981
Feb. 1, 1924	............Membership............	1,161
Feb. 1, 1925	............Membership............	1,425
Feb. 1, 1926	............Membership............	1,774

—not a spectacular growth,—a consistent, steady climb.

In February 139 new members were added; in March 44. Losses by resignation bring the total April 1 to 1,944.

July 1, 1925, we had 1,426 active members and 110 supporting members (following losses in May and June when delinquents were dropped from the roll).

The budget for the fiscal year July 1, 1925, to June 30, 1926, was based on 1,700 active members and 125 supporting members—less an estimated loss of 10 per cent by resignation and delinquency. (10 per cent is normal loss.)

April 1, 1926, the membership of the Institute is 1,828 active and 116 supporting.

Against this, costs have exceeded estimates for preprints and other printing.

The present situation will be covered by proper audit in July, 1926, following the close of the present fiscal year.

Appended is the report of the Meissner Audit Company for the fiscal year ended June 30, 1925. It shows an abnormal expenditure for Proceedings—nearly all of the expense for Volumes 20 and 21 having been paid in the fiscal year covered by the report because Volume 20 was not issued until after the close of the fiscal year in which it belonged.

A brief statement then of the financial situation June 30, 1925, as compared with June 30, 1924, is as follows:

June 30, 1924, we had Cash in Bank	$2,535.00
In Treasury Certificates	7,500.00
In Imprest Cash	300.00
	$10,335.00
Against that Unpaid Bills for Vol. 20 Proceedings	5,928.37
Leaving a Cash Surplus of	$4,406.63
June 30, 1925, we had Cash in Bank	$745.00
In Treasury Certificates	4,500.00
In Imprest Cash	300.00
	$5,545.00
Unpaid Bills	None
An Increase in Cash Surplus of	$1,138.37

MEISSNER AUDIT COMPANY
DETROIT, MICH.

DETROIT, MICH., August 21, 1925.

Mr. Harvey Whipple, Treasurer,
American Concrete Institute,
1807 East Grand Boulevard,
Detroit, Michigan.

DEAR SIR:

In accordance with your request we have Audited the Cash Received and Disbursed from July 1, 1924, to June 30, 1925, as shown by your Books, and have found that all Cash reported received has been properly deposited in the National Bank of Commerce of this City, also that all Disbursements are represented by proper Vouchers.

The Bank Account has been verified and agrees with Statement supplied by your Bank.

No attempt has been made to verify Cash Receipts reported, with your membership.

Thanking you for the courtesy and assistance given us during the course of our Audit,

Yours respectfully,

MEISSNER AUDIT COMPANY,

J. W. MEISSNER.

AMERICAN CONCRETE INSTITUTE.
BALANCE SHEET.
June 30, 1925.

Assets.

Cash:		
On Hand	$300.00	
In Bank	745.60	
Total Cash		$1,045.60
U. S. 4½ Per Cent Treasury Certificates		4,500.00
Accounts Receivable:		
Active Members (161 @ $10.00 and 1 @ $6.00)	$1,616.00	
Contributing Members (2 @ $50.00)	100.00	
Miscellaneous	777.53	
Total Accounts Receivable		2,493.53
Inventories:		
408—1914-15 Journals @ 10 cents each	$40.80	
409—1905 to 1919 Proceedings @ 50 cents each	204.50	
589—1919 to 1923 Proceedings @ $1.00 each	589.00	
122—1924 Proceedings @ $3.50 each	427.00	
Total Inventories		1,261.30
Total Assets		$9,300.43

Liabilities.

Deferred:		
Dues Paid in Advance	$150.00	
Reserve:		
For Loss Due to Delinquent Members	1,700.00	
Surplus:		
Deposited by Members	7,450.43	
Total Liabilities		$9,300.43

AMERICAN CONCRETE INSTITUTE.
RECEIPTS AND DISBURSEMENTS.
July 1, 1924, to June 30, 1925.

Cash on Hand, July 1, 1924		$2,535.00
RECEIPTS.		
Dues Active	$13,563.50	
Dues Contributing	5,310.00	
Preprint Sales	2,910.09	
Proceeding Sales	1,191.38	
Interest Earned	478.91	
U. S. Treasury Certificates	3,000.00	
Total Receipts		26,453.88
		$28,988.88
DISBURSEMENTS.		
Convention	$1,140.03	
Exchange	.30	
Membership, National Fire Protection Association	60.00	
Auditing	45.00	
Secretary's Bond	50.00	
Miscellaneous	54.59	
Preprints	4,209.21	
Office Expense	369.04	
Postage	816.63	
Printing, Multigraphing, Etc.	3,101.52	
Proceedings	11,402.03	
Rent	379.92	
Salaries	6,327.90	
Traveling	343.20	
	$28,299.37	
Less—Discounts, Net	56.09	
Total Disbursements		28,243.28
Cash in Bank June 30, 1925		$745.60

AMERICAN CONCRETE INSTITUTE.
BANK RECONCILIATION.
June 30, 1925.

Balance—As per Ledger		$745.60
Add—Checks Outstanding:		
Number 2060	$100.00	
Number 2061	221.00	
Number 2062	8.88	
Number 2063	9.39	
Number 2064	300.00	
Number 2065	4,314.60	
Number 2066	31.69	
Number 2067	16.50	
Number 2068	358.30	
Total Outstanding Checks		5,360.46
Balance in Bank, June 30, 1925		$6,106.06

LIST OF REGISTRANTS.

ABRAMS, DUFF A., Structural Materials Research Laboratory, Chicago.
AFFLECK, B. F., Universal Portland Cement Co., Chicago.
AHLERS, JOHN G., Barney-Ahlers Constr. Corp., New York.
ALLEN, J. LLOYD, California Stucco Prod. Co., Indianapolis, Ind.
ALLEN, LESLIE H., Concrete Tile Machy. Co., Cicero, Ill.
ALLEN, O. T., American Steel & Wire Co., Chicago, Ill.
ALLEN, M. S., Youngstown Pressed Steel Co., Warren, Ohio.
ALBRECHT, RALPH W., Plastic Products Co., Milwaukee, Wis.
AHEARN, V. P., National Sand & Gravel Assn., Washington, D. C.
ALLEN, R. E., W. N. Best Corp., New York.
ALLEN, W. D. M., Portland Cement Association, Chicago, Ill.
ALEXANDER, E. C., Massey Concrete Prod. Corp., Chicago.
AMAR, V. F., Core Joint Concrete Pipe Co., Irvington, N. J.
ANDERSON, LOUIS, JR., Alpha Portland Cement Co., Easton, Pa.
ANDERSON, W. P., The Ferro Concrete Constr. Co., Cincinnati, Ohio.
ARNOLD, M. A., Arnold Brick & Tile Co., Jacksonville, Fla.
ARTHUR, G. B., National Lime Association, Washington, D. C.
ARMS, L. M., Portland Cement Association, Chicago, Ill.
ASHTON, ERNEST, Allentown, Pa.
BACHMAN, P. R., Y. B. S. Co., Warren, Ohio.
BALDRIDGE, C. W., A. T. & S. F. Ry., Chicago, Ill.
BALDWIN, C. M., R. D. Kinder Co., Chicago, Ill.
BALL, CHAS. B., Health Dept., City Hall, Chicago, Ill.
BALL, CHAS. F., Chain Belt Co., Milwaukee, Wis.
BECK, ALEXANDER, W. N. Best Corp., New York, N. Y.
BARRIBALL, GEORGE D., Cleveland, Ohio.
BATEMAN, JOHN H., Louisiana State Univ., Baton Rouge, La.
BARREN, K. A., Cleveland, Ohio.
BASS, J. FREDERIC, Univ. of Minn., Minneapolis, Minn.
BATES, P. H., Washington, D. C.
BASTIANI, B., Plastic Products Co., Milwaukee, Wis.
BEGGS, NORMAN, Portland Cement Association, Chicago, Ill.
BEARD, FREDERICK S., The Ohio Hydrate & Supply Co., Detroit, Mich.
BERCHEM, H. C., St. Paul Cement Works, St. Paul, Minn.
BERNIER, N. M., California Stucco Co. of New England, Cambridge, Mass.
BENSON, NEWTON D., Providence, R. I.
BELLE, H. W., Atlas Cement Co., Chicago, Ill.
BISWANGER, S. J., Chicago, Ill.
BJORNDAKL, RICHARD, Moline Cast Stone Co., Moline, Ill.
BOGUE, B. H., Portland Cement Association, Washington, D. C.
BLAINE, ETHEL E., Ass't Sec'y, American Concrete Institute, 2970 West Grand Blvd., Detroit, Mich.
BORGO, N. G., N. G. Borgo Co., Darian, Wis.
BOSCH, JACOB, 100 W. 115th St., Chicago, Ill.
BOUILLON, A. M., Illinois Central R. R., Chicago, Ill.
BOURNE, C. L., Portland Cement Association, Chicago, Ill.
BOWDITCH, JOHN, JR., Youngstown Pressed Steel Co., Warren, Ohio.
BOWLING, R. M., The Bettendorf Co., Bettendorf, Iowa.
BOYD, H. W., Youngstown Pressed Steel Co., Dayton, Ohio.
BOYDEN, H. C., COL., Ohio Northern University, Ada, Ohio.
BOYER, E. D., Atlas Portland Cement Co., New York, N. Y.

BRADSHAW, GEORGE L., Independent Block & Cement Co., Indianapolis, Ind.
BRANDTZAAG, ANTON, University of Ill., Urbana, Ill.
BRAGGER, E. J., The Sandusky Cement Co., Cleveland, Ohio.
BRASSERT, W. O., Michigan Silo Co., Kalamazoo, Mich.
BRAUW, WALTER, Columbus, Ohio.
BREWER, N. L., Brewer Bros. & Rose, Huntington, W. Va.
BREWER, R. D., Portland Cement Association, Chicago, Ill.
BROOKMAN, LOUIS, JR., 139 N. Clark St., Chicago, Ill.
BROKER, A. E., Badger Cement Tile Co., Plymouth, Wis.
BROOKS, FRANK M., Brooks Art Stone Corporation, Pasadena, Calif.
BROWN, F. E., Holabird & Roche, Chicago, Ill.
BROWN, H. E., Milwaukee, Wis.
BROWN, P. V., Robt. W. Hunt, Chicago, Ill.
BROWN, REX L., University of Ill., Urbana, Ill.
BROWN, R. P., National Lime Association, Washington, D. C.
BROWNE, H. R., Huron Portland Cement Co., Detroit, Mich.
BRUNER, BERNARD L., California Stucco Products Co., Cincinnati, Ohio.
BUDD, CHAS. A., Dept. Public Works, Chicago, Ill.
BUENTE, C. F., Concrete Products Co. of America, Pittsburgh, Pa.
BUCHHOLZ, H. E., Oak Park, Ill.
BULLEN, CARROLL A., Portland Cement Association, Chicago, Ill.
BULLEN, C. H., Concrete Pipe Co., Portland, Ore.
BUTLER, C. M., Marquette Cement Manufacturing Co., LaSalle, Ill.
BURGESS, S. W., National Slag Association, Cleveland, Ohio.
BURKHOLDER, PAUL C., The Practical Cement Block Co., Indianapolis, Ind.
BURT, H. J., 104 S. Michigan Ave., Chicago, Ill.
BUTLER, GEO. A., L. M. Ludington's Sons, Rochester, N. Y.
BUTLER, MORGAN R., Butler Bin Co., Waukesha, Wis.
CAMP, LEON K., Concrete Products Co., Columbus, Ga.
CAPOUCH, M. E., Portland Cement Association, Chicago, Ill.
CARLSON, A. G., Universal Portland Cement Co., Chicago, Ill.
CARNOOHAN, D. E., National Lime Association, St. Louis, Mo.
CARREL, F. G., Calumet Steel Co., Chicago, Ill.
CASE, C. A., Cook County Highway Department, Chicago. Ill.
CASE, R. M., W. J. Hynes, Ltd., Toronto, Ont., Can.
CATLETT, RICHARD H., North American Cement Corp., Hagerstown, Md.
CHANDLER, GEO. D., Superior Sand & Gravel Co., Detroit, Mich.
CHAPMAN, CLOYD M., Dwight P. Robinson, New York.
CHILL, CHAS. W., Detroit Edison Co., Detroit, Mich.
CHIPMAN, PAUL, Pere Marquette Ry., Detroit, Mich.
CHITTENDEN, HOWARD L., Clinton, Conn.
CHUBB, JOS. H., Universal Portland Cement Co., Chicago, Ill.
CLARKE, H. V., The Philip Carey Co.
CLOKE, A. D., Youngstown Pressed Steel Co., Warren, Ohio.
CLEMMER, H. F., Solvay Process Co., New York.
CLEVE, ALBERT, Structural Materials Research Laboratory, Chicago, Ill.
COGHLAN, RAPIER R., Southwestern Portland Cement Co., Osborn, Ohio.
COHEN, A. B., Consulting Engr., New York.
COLBURN, D. S., Marquette Cement Mfg. Co., Chicago, Ill.
COLLINS, D. R., Concrete Products, A. W. Friske, Alois, Wis.
COLMAR, DANIEL, Albany, N. Y.
CONDRON, T. L., Condron & Post, Chicago, Ill.
CONNER, R. S., Signal Mountain Portland Cement Co., Chattanooga, Tenn.
CONRADES, OTTO S., St. Louis Material & Supply Co., St. Louis, Mo.
CONOVER, GEORGE, Contractor's & Engineer's Monthly, Chicago, Ill.
CORBIN, MARGARET ARRONET (MRS.), Struct. Mat. Research Lab., Chicago, Ill.
COOKE, CHAS. E., Kalman Floor Co., Chicago, Ill.

COX, ORA C., Youngstown Steel Products Co. (Youngstown, Ohio), Evanston, Ill.
CRABBS, AUSTIN, The Cement Products Co., Davenport, Iowa.
CRAY, HOWARD A., Morton C. Tuttle Co., Boston, Mass.
CREPPS, R. B., Purdue University, West LaFayette, Ind.
CREW, S. I., Norwood Concrete Block Con. Co., Norwood, Ohio.
CROSS, S. P., Universal Gypsum Co., Chicago, Ill.
CROCKER, IRVING G., Youngstown Steel Products Co., Chicago, Ill.
CROSBY, E. S., Celite Products Co., New York.
CRUM, R. W., Iowa Highway Com., Ames, Iowa.
CUMMINGS, H. P., Municipal Research Bureau, Cleveland, Ohio.
CUNNICK, PAUL C., Rock Island Arsenal.
CURREY, F. W., Selden-Breck Constr. Co., Omaha, Neb.
CURTIS, A. J. R., Portland Cement Association, Chicago, Ill.
DAMON, A. F., JR., Upper Darby, Pa.
DAVIS, E. E., E. E. Davis Co., Chicago, Ill.
DEBERARD, W. W., *Engineering News Record*, Chicago, Ill.
DE CAMP, H. I., C. B. & Q. R. R., Chicago, Ill.
DE CAMP, L. E., Sandusky Cement Co., Dixon, Ill.
DEINBOLL, F K., N. Y. Lines West, Cleveland, Ohio.
DECKER, DAVID A., Lock Joint Pipe Co., Chicago, Ill.
DEVOS, A. W., Wm. H. Devos Co., Milwaukee, Wis.
DEVER, CLARE, Huron Portland Cement Co., Detroit, Mich.
DEINLEIN, EMIL E., Home Building Supply Co., Milwaukee, Wis.
DIECKMEYER, L. M., St. Louis, Mo.
DICKERSON, O. H., Duluth & Iron Range R. R., Duluth, Minn.
DIXON, D. H., Turner Construction Co., New York, N. Y.
DIENHART, E. W., Acme Concrete Prod. & Gravel Co., Cement City, Mich.
DONOHUE, JERRY, Sheboygan, Wis.
DOCKSTADER, E. A., Stone & Webster, Inc., Boston, Mass.
DOUTHETT, C. L., Waterloo Concrete Corp., Waterloo, Iowa.
DOUGHERTY, G. H., D. & A. Prost Mold Co., Three Rivers, Mich.
DREISBACH, EDW. E., Nazareth Cement Co., Nazareth, Pa.
DROLENGA, N. C., Supt. of Const., Bd. of Ed., Chicago, Ill.
DUGAN, CHAS. B., National Steel Fabric Co., Chicago, Ill.
DUNDAS, F. C., Portland Cement Association, Chicago, Ill.
DUNHAM, H. L., Celite Products Co., Chicago, Ill.
DUNNELLS, C. G., Hunting, Davis & Dunnells, Pittsburgh, Pa.
DURGIN, F. L., JR., Cramp & Co., Philadelphia, Pa.
DURYEA, L. N., Cowham Eng. Co., Chicago, Ill.
DUWE, EDWARD, Badger Concrete Co., Oshkosh, Wis.
EADES, H. H., National Steel Fabric Co.
EARLEY, JOHN J., Washington, D. C.
EDLUND, LAWRENCE L., Gardner & Lindberg, Chicago, Ill.
EDWARDS, C. E., Lamar Pipe and Tile Co., Grand Rapids, Mich.
EHLERT, E. H., Cleveland, Ohio.
ELWELL, J. S., National Lime Association, Washington, D. C.
EGE, C. R., Portland Cement Association, Chicago, Ill.
EMERSON, H. B., Lehigh Portland Cement Co., Chicago, Ill.
ENBLOM, ALBERT, Nelson-Enblom Co., Minneapolis, Minn.
ENGLAND, ERNEST W., United Fuel & Supply Co., Detroit, Mich.
EPPES, N. A., Gulf Concrete Pipe Co., Houston, Texas.
EVANS, T. ARTHUR, Board of Local Improvements, Chicago, Ill.
EVANS, HARRY G., Louisville, Ky.
FARMER, H. G., Universal Cement Co., Pittsburgh, Pa.
FEATHER, PRESTON, Petoskey, Mich.
FEHLIG, EDWARD, Fehlig-Ferrenbach, Inc., St. Louis, Mo.
FENKEL, STANLEY O., Penn. Bldg. Block Co., Philadelphia, Pa.

Fensel, Alden C., Municipal Research Bureau, Cleveland, Ohio.
Ferrenbach, E. C., Fehlig-Ferrenbach, Inc., St. Louis, Mo.
Ficker, Frank B., The Youngstown Pressed Steel Co., Warren, Ohio.
Fischer, W. G., 5114 Washington Blvd., Chicago, Ill.
Fixen, V. L., R. J. McLeod & Co., Duluth, Minn.
Flam, Stephen, Supertile Mach. Corp., Los Angeles, Calif.
Fletcher, R. C., Flint Crushed Gravel Co., Des Moines, Iowa.
Flodin, H. L., Portland Cement Association, Chicago, Ill.
Floyd, George F., Turner Construction Co., New York, N. Y.
Foley, R. A., Superior Sand & Gravel Co., Detroit, Mich.
Ford, Robert H., Rock Island Lines, Chicago, Ill.
Forrest, V. E., Bland Engineering Co., Minneapolis, Minn.
Foshinbaur, V. G., Portland Cement Association, Chicago, Ill.
Foster, C. B., Foster Engr. Service, Indianapolis, Ind.
Fournie, Arthur J., Swansea Stone Works, East St. Louis, Ill.
Francis, Carl, Phoenix Portland Cement Co., Philadelphia, Pa.
Franklin, Jack, Anchor Concrete Machinery Co., Chicago, Ill.
Freeman, J. E., Par-Lock Appliers of Chicago, Chicago, Ill.
Freeman, P. J., Allegheny Co. D. P. W., Pittsburgh, Pa.
Friel, G. M., Anchor Con. Machy. Co., Columbus, Ohio.
Fugate, G. L., 207 City Hall, Houston, Texas.
Fulhuson, S. C., Handy Sack B. Co., Marion, Iowa.
Gallett, C. O., New York Concrete Pipe Co., Wassaw, N. Y.
Gaston, W. G., Atlas Concrete Units Co., Blue Island, Ill.
Gauckler, A. J., Milwaukee School Board, Milwaukee, Wis.
Gardnier, Lion, Lakewood Engineering Co., Cleveland, Ohio.
Gebhart, B. R., Portland Cement Association, Chicago, Ill.
Gehman, Charles, Wasser Concrete Products Corp., New York, N. Y.
Gilkey, Herbert J., University of Colorado, Boulder, Colo.
Ginder, J. W., Washington, D. C.
Ginsberg, Frank I., Ginsberg Penn Co.
Giryotas, W. J., R. W. Hunt Co.
Gochnauer, C. O., Gochnauer Concrete Prod. Co., Appleton, Wis.
Goldie, Wm., Jr., Goldie Mfg. Corp., Pittsburgh, Pa.
Going, H. C., The Going Road Machinery Co., Birmingham, Ala.
Gonnerman, H. F., Structural Materials Research Laboratory, Chicago, Ill.
Goldbeck, A. T., National Crushed Stone Association, Washington, D. C.
Goodman, E. M., Evanston, Ill.
Gosswein, O. H., Universal Portland Cement Co., Chicago, Ill.
Goodwin, P. M., The Solvay Process Co., Chicago, Ill.
Grady, Joseph C., Turner Construction Co., Chicago, Ill.
Graham, R. J., H. L. Stevens & Co., Chicago, Ill.
Graham, Harold L., Dixie Concrete Products Co., Chattanooga, Tenn.
Grandy, A. L., Pere Marquette Ry., Detroit, Mich.
Graham, John C., Jackson, Mich.
Griffin, H. E., Besser Sales, Chicago, Ill.
Griffith, E. A., Allegheny Co. Road Dept., Pittsburgh, Pa.
Grube, L. E., Sheboygan Brick Co., Sheboygan, Wis.
Gurtler, Wm. A., James O. Heyworth, Inc., Chicago, Ill.
Guy, Frank E., Universal Portland Cement Co., Pittsburgh, Pa.
Haddon, Hugh, Jr., Menantico Sand & Gravel Co., Rockaway, N. J.
Hadley, H. M., Portland Cement Association, Seattle, Wash.
Haegert, L. V., A. T. & S. F. R. R., Topeka, Kan.
Haight, E. E., Concrete Publishing Co., Chicago, Ill.
Hall, John W., Hugh L. Cooper & Co., New York, N. Y.
Halliday, H. H., Halliday Sand Co., Cairo, Ill.
Halloran, Paul J., U. S. N., Great Lakes, Ill.

HAMMOND, A. J., Evanston, Ill.
HANSARD, O. H., State Highway Department, Nashville, Tenn.
HANSON, E. S., 542 Monadnock Block, Chicago, Ill.
HARPER, GEO. L., Chicago Union Station Co., Chicago, Ill.
HARRIS, A. C., Midland Supply & Coal Co., Alton, Ill.
HARDY, CROSS, University of Illinois, Urbana, Ill.
HARRIS, JOHN BYRON, National Lime Association, Chicago, Ill.
HARRIS, T. J., Portland Cement Association, Chicago, Ill.
HARRIS, WALLACE R., The Eberling Machines Sales Co., Oak Park, Ill.
HARRISON, MERRITT, Harrison & Turnock, Indianapolis, Ind.
HARRISON, J. L., Bureau of Public Roads, Washington, D. C.
HART, W. E., Portland Cement Association, Chicago, Ill.
HARTY, JOHN J., Monks & Johnson, Boston, Mass.
HARZA, L. F., Monadnock Building, Chicago, Ill.
HATT, K. A., Concrete Publishing Co., Chicago, Ill.
HATT, W. K., Purdue University, LaFayette, Ind.
HAUSER, GEORGE R., Comm. of Buildings, Cincinnati, Ohio.
HAVLIK, ROBT. F., Engr. & Mfg. of trim stone, Mooseheart, Ill.
HAWK, LESTER C., Dexter Portland Cement Co., Nazareth, Pa.
HAY, WM. WREN, Mack International Motor Co.
HEALEY, A. S., Glencoe Lime & Cement Co., St. Louis, Mo.
HEALY, J. T., Atlas Portland Cement Co., Chicago, Ill.
HECK, GEO. H., Pennsylvania Brick & Tile Co., Philadelphia, Pa.
HECK, HERMAN H., Louisville Gas & Electric Co., Louisville, Ky.
HEEB, E., Standard Concrete Pipe & Curb Co., New Orleans, La.
HELLER, MARTIN, Concrete Contr., Granite City, Ill.
HENDRICKS, P. J., Northern Lime & Stone Co., Petoskey, Mich.
HENDY, CHARLES, JR., American Hume Concrete Pipe Co., Denver, Colo.
HERZOG, C. J., Secretary, Cretestone Builders' Supply Co., Pittsburgh, Pa.
HICKOX, J. R., C. B. & Q. R. R., Chicago, Ill.
HINDMAN, MARION K., MISS, J. E. Adams, Johnstown, Pa.
HINDMAN, W. S., Columbus, Ohio.
HIRSCHBERG, WALTER, Federal Engineering Co., Milwaukee, Wis.
HITCHCOCK, FRANK A., Bureau of Standards, Washington, D. C.
HOGG, W. T., Dock Board, New Orleans, La.
HOHL, E. E., James O. Heyworth, Inc., Chicago, Ill.
HOLINGER, ARNOLD C., Tait & Lord, Inc., Chicago, Ill.
HOLLINGSHEAD, T. H., Zion Inst. & Industries, Zion, Ill.
HOLLISTER, S. C., Consulting Engineer, Philadelphia, Pa.
HOLM, W. M., A. T. & S. F. Ry., Amarillo, Texas.
HOOD, J. H., Brooks Art Stone Corp., Pasadena, Calif.
HORST, A. E., Rock Island, Ill.
HOWARD, WM. F., Portland Cement Association, Chicago, Ill.
HOWE, HENRY L., Engr. Dept., Rochester, N. Y.
HOWE, H. N., Gardner & Howe, Memphis, Tenn.
HUBBARD, FRED, The Standard Slag Co., Youngstown, Ohio.
HUMPHREY, RICHARD L., Consulting Engr., Philadelphia, Pa.
HUSSEY, GEO. A., Brussel & Viterbo, Chicago, Ill.
HUSSEY, JACK W., Atlas Portland Cement Co., Chicago, Ill.
HUTCHINSON, G. W., Raleigh, N. C.
IMMEL, J. W., Immel Const. Co., Fond du Lac, Wis.
INGERMANSON, T. W., Chicago, Ill.
IRWIN, A. C., 111 W. Washington St., Chicago, Ill.
JACKSON, FRANK H., Bu. Pub. Roads, Washington, D. C.
JAECKLE, JULIUS, Jaeckle's Cement Block, Okauckee, Wis.
JAFFE, S. R., Board of Education, Chicago, Ill.
JAMES, J. R., Detroit Edison Co., Detroit, Mich.

JENNINGS, D. F., Concrete Publishing Co., Chicago, Ill.
JERDEE, ALBERT C., Zion, Ill.
JOHANNES, G. H. F., Glencoe Lime & Cement Co., St. Louis, Mo.
JOHNSON, A. C., Union Oil Co., Los Angeles, Calif.
JOHNSON, CARL O., Brdg. Div., Chicago, Ill.
JOHNSON, C. S., C. S. Johnson Co., Champaign, Ill.
JOHNSON, O. A., R. W. Hunt Co., Chicago, Ill.
JOHNSON, ROBERT C., Immel Construction Co., Fond du Lac, Wis.
JOHNSON, T. H., Consulting Engr., Sioux City, Iowa.
JOHNSON, VIRGIL L., Germantown, Philadelphia, Pa.
JOHNSON, WM. R., Structural Materials Research Laboratory, Chicago, Ill.
JONES, W. C., Roseland Concrete Products Co., Chicago, Ill.
JOSSOY, WILLIAM A., Heine Chimney Co., Chicago, Ill.
KAISER, W. G., Portland Cement Association, Chicago, Ill.
KALINKAS, J. E., Roberts & Schaeffer Company, Chicago, Ill.
KEARNEY, JAMES C., Detroit, Mich.
KEATINGE, PAUL F., Atlas Portland Cement Co., Chicago, Ill.
KEITH, C. W., C. B. & Q. R. R., Chicago, Ill.
KENNEDY, J. W., Portland Cement Association, Chicago, Ill.
KIMMEL, W. D., Amer. Hume Concrete Pipe Co., Detroit, Mich.
KING, A. W., Atlas Portland Cement Co., Chicago, Ill.
KING, G. C., Spencer Cement Block Works, Spencer, Iowa.
KINNEY, W. M., Portland Cement Association, Chicago, Ill.
KLINGENSMITH, F. L., Amer. Hume Concrete Pipe Co., Detroit, Mich.
KOLB, F. X., Detroit, Mich.
KOSANKE, PAUL L., Plastic Products Co., Milwaukee, Wis.
KREHBIEL, B. F., Cement Stone & Supply Co., Wichita, Kan.
KRIER, GEO. H., Concrete Products Mfr., Brooklyn, N. Y.
KVITRUD, Contr.-Engr., Minneapolis, Minn.
LACKEY, H. W., Potter Const. Co., Chicago, Ill.
LA DUE, R. M., McCracken Machinery Co., Sioux City, Iowa.
LAKER, H. J., Laker Concrete Works, Dalton, Mo.
LANDER, R. S., Shearman Concrete Pipe Co., Little Rock, Ark.
LANDRY, JOHN J., Rock Products, Chicago, Ill.
LANG, J. E., Raber & Lang Mfg. Co., Kandallville, Ill.
LAPHAM, JOHN R., George Washington University, Washington, D. C.
LAPIERRE, C. C., Promotion Engineer, Montreal, Can.
LARKIN, CHAS. W., Carnegie Institute of Tech., Pittsburgh, Pa.
LARKIN, EDWARD C., Anchor Concrete Machinery Co., Dayton, Ohio.
LEACH, FRED M., Anchor Concrete Machinery Co., Detroit, Mich.
LEAVITT, H. WALTER, University of Maine, Orono, Me.
LEE, R. F., Central Concrete Co., Indianapolis, Ind
LEHMAN, CHESTER H., Blow Knox Co., Pittsburgh, Pa.
LEVINSON, ARTHUR A., Blow Knox Co., Pittsburgh, Pa.
LEWIS, W. A., Concrete Products Corp., Milwaukee, Wis.
LIBBERTON, J. H., General Chemical Co., New York, N. Y.
LICHTENBERG, E. H., Koehring Co., Milwaukee, Wis.
LILLIE, G. F., Platte Valley Cement Tile Mfg. Co., Fremont, Neb.
LINDAU, A. E., Chicago, Ill.
LINDSEY, GEO. L., Universal Portland Cement Co., Chicago, Ill.
LINDSTROM, ROBERT SETH, 203 S. Dearborn St., Chicago, Ill.
LINGENFELTER, H. W., Westinghouse, South Bend, Ind.
LIVERMORE, A. C., W. A. B. Home Building Co., Wilmerding, Pa.
LIVINGSTON, J. A., Granite Concrete Block Co., Toronto, Ont.
LOCKHARDT, WILLIAM F., New York, N. Y.
LONGFELLOW, D. W., Elk River Concrete Products Co., Elk River, Minn.
LORD, ARTHUR R., Tait & Lord, Chicago, Ill.

LORD, C. L., Portland Cement Association, Chicago, Ill.
LOVE, H. J., National Slag Association, Cleveland, Ohio.
LOUCKS, H. F., Ideal Concrete Machinery Co., Chicago, Ill.
LOVEJOY, C. H., Forest City Testing Lab., Cleveland, Ohio.
LOWELL, JOHN W., Benedict Stone Wks., Chicago, Ill.
LYONS, JOHN J., Best Bros., Chicago, Ill.
LURIE, E. M., Associated Metal Lath Mfrs., Chicago, Ill.
MCCLATCHY, JOHN H., Builder, Philadelphia, Pa.
MCCULLOUGH, F. M., Carnegie Institute of Technology, Pittsburgh, Pa.
MCCULLY, H. M., Carnegie Inst. Tech.
MCKAY, EARLE D., Universal Portland Cement Co., Minneapolis, Minn.
MCKEE, S. C., Truscon Steel Co., Toledo, Ohio.
MCLAUGHLIN, F. C., Kent Machine Co., Kent, Ohio.
MCMILLAN, F. R., Portland Cement Association.
MCNIVEN, J. A., Dock Board, New Orleans, La.
MCQUIDE, J. C., Blaw-Knox Co., Chicago, Ill.
MABIE, HARRY W., JR., Fond du Lac, Wis.
MACGOWAN, E. S., Universal Portland Cement Co., Minneapolis, Minn.
MACLIND, R. J., Duluth, Minn.
MADOLE, WARD W., Des Moines, Iowa.
MAGEE, C. L. C., Octagonal Reinforced Floor Co., Chicago, Ill.
MAGEE, W. H., National Lime Association, Chicago, Ill.
MAHAN, M. J., Midland Valley Coal & Mat. Co., Overland, Mo.
MALANEY, J. W., Huron Cement Co., Detroit, Mich.
MALONE, O. A., Calif. Stucco Products Co., Los Angeles, Calif.
MANY, BEN J., Western Waterproofing Co., St. Louis, Mo.
MARKLAND, MATTHEW B., Atlantic City, N. J.
MARSHALL, R., Advertising, Detroit, Mich.
MARSH, DON E., Windes & Marsh, Winnetka, Ill.
MARTIN, G. E., Bessemer and Lake Erie R. R., Conneaut, Ohio.
MARTIN, S. G., Universal Cement Co., Chicago, Ill.
MATTHEWS, A. A., Western Concrete Products Co., Denver, Colo.
MAURER, E. R., University Wisconsin, Madison, Wis.
MEADE, S. J., Heine Chimney Co., Chicago, Ill.
MEERMANS, H. G., The Youngstown Pressed Steel Co., Warren, Ohio.
MENDENHALL, F. B., Penna. Railroad, Chicago, Ill.
MENZEL, C. A., Underwriters Lab., Homewood, Ill.
MERRIMAN, THADDEUS, New York, N. Y.
MEECH, FRANK M. G., St. Joseph, Mich.
MEYER, B. A., Meyer, Morrison & Co., Inc., New York, N. Y.
MEYER, WM. H., Immel Construction Co., Fond du Lac, Wis.
MILLER, FREDERICK A., Product, Hagerstown, Md.
MILLER, ALLAN O., O. S. Miller & Co., Indianapolis, Ind.
MILLER, HUGH P., The Michigan Silo Co., Peoria, Ill.
MILLER, J. E., Milledgsville, Ill.
MINER, JOSHUA L., Atlas Lumnite Cement Co., New York, N. Y.
MOORE, O. L., Universal Portland Cement Co., Chicago, Ill.
MOORE, S. P., Cement Products, Cedar Rapids, Iowa.
MORFORD, G. O., Roberts & Schaefer Co., Berwyn, Ill.
MORRIS, MARK, Iowa State Highway Comm., Ames, Iowa.
MORRISON, R. L., Mich. State Highway Department, Ann Arbor, Mich.
MUIR, F., C. E. American Bridge, Gary, Ind.
MURPHY, T. W., Youngstown Pressed Steel Co., Detroit, Mich.
MYLREA, T. D., Urbana, Ill.
NELLES, J. S., Detroit Edison Co., Detroit, Mich.
NEVINS, H. G., Sandusky Cement Co., Chicago, Ill.
NEWBERRY, WM. B., Sandusky Cement Co., Cleveland, Ohio.

NICHOLS, CHARLES E., Stone & Webster, Inc., Boston, Mass.
NICKERSON, BLAKE L., Livonia Cement Block Fcty., Livonia, N. Y.
NOBLE, THOS. W., Thos. W. Noble & Co., Chicago, Ill.
NOLAND, C. J., Chicago Union Sta., Chicago, Ill.
NORRIS, J. F., 157 Yarmouth Road.
O'BRIEN, M. J., Underwriters Laboratories, Chicago, Ill.
OEHRLE, WILLIAM, Longnere Eng. & Const. Co., Chicago, Ill.
OLSEN, EDWARD, Badger Concrete Co., Oshkosh, Wis.
OLSEN, EUGENE F., Anchor Concrete Machinery Co., Adrian, Mich.
OTT, H. W., Concrete Products Corp., Milwaukee, Wis.
OTTO, E. D., Downers Grove, Ill.
OWEN, GEORGE, Zion, Ill.
PADDOCK, W. F., Peerless Concrete Products Co., Seattle, Wash.
PARKER, HENRY, Co., Hollywood Bldg. Block Co., Allentown, Pa.
PATZIG, MONROE L., Patzig Laboratories, Des Moines, Iowa.
PAULSON, HAAKON, Anchor Concrete Machinery Co., Minneapolis, Minn.
PEARSON, J. C., Lehigh Portland Cement Co., Allentown, Pa.
PEASE, B. S., American Steel & Wire Co., Chicago, Ill.
PECK, ROY L., Portland Cement Assn., Chicago, Ill.
PECKHAM, F. B., Lakewood Engineering Co., Chicago, Ill.
PITNER, L. E., Concrete Products Co., Milwaukee, Wis.
PIERCE, CLYDE, Indianapolis, Ind.
POLANSKY, THEO. A., Union Concrete Pipe Co., Huntington, W. Va.
POLLARD, ALBERT A., Chicago, Ill.
POPENHAGEN, F. M., Concrete Products Co., Cedar Rapids, Iowa.
PORTER, G. W., Universal Portland Cement Co., Waukesha, Wis.
POTTER, A. W., Zion Inst. & Industries, Zion, Ill.
POTTS, H. H., Crystal Bldg. Products Corp., Chicago, Ill.
POWELL, F. H., Southwestern Portland Cement Co., Los Angeles, Calif.
POWERS, JOHN M., Sterling, Ill.
PRARD, JOHN G., Gibraltar Cement Block Co., Indianapolis, Ind.
PRIEST, GEORGE E., Rib-Stone Concrete Corp., LeRoy, N. Y.
PRIEST, MRS. GEORGE E., LeRoy, N. Y.
PRIMAR, JOHN, Stewart Sand Co., Kansas City, Mo.
RANDALL, FRANK A., Chicago, Ill.
RANSOHOFF, NATHAN, Ideal Concrete Mchy. Co., Cincinnati, Ohio.
RAYMOND, CHAS., Montreal, Can.
RAYMOND, MAURICE, Raymond Cement Products Co., Montreal, Que., Can.
REED-LEWIS, E. W., Super Cement Co., Toronto, Can.
REMLER, R. F., Grasselli Chemical Co., Cleveland, Ohio.
RICE, JOHN A., Bubblestone Co., Berkeley, Calif.
RICHART, F. E., University of Illinois, Urbana, Ill.
RIDDING, G. H., Massey Conc. Prod. Corp., Chicago, Ill.
RIDGE, R. S., California Stucco Prod. Co. of Am., Philadelphia, Pa.
RITZERT, A. L., Naperville, Ill.
RITZERT, W. H., Naperville, Ill.
ROBERTS, C. A., Galien, Mich.
ROBERTS, WM. H., Rochester, N. Y.
ROBERTSON, R. D., Independent Concrete Pipe Co., Indianapolis, Ind.
ROBERTSON, W. H., Massey Conc. Prod. Co., Chicago, Ill.
ROBINSON, H. H., St. Louis, Mo.
ROBINSON, HUGH M., Underwriters Laboratories, Chicago, Ill.
ROGERS, H. A., A. B. Keeport Co., Inc., Indianapolis, Ind.
ROQUERMORE, J. D., Roquemore Gravel Co., Montgomery, Ala.
RUBLING, J. A., Atlas Lumnite Cement Co., Chicago., Ill.
SACKL, GUST. H., Plastic Products Co., Milwaukee, Wis.
SAEGER, GEOFF A., Crescent Portland Cement Co., Wampum, Pa.

SAFFERT, GEORGE, New Ulm, Minn.
SEABURY, GEORGE T., Secy., Amer. Soc. of Civil Engineers, New York, N. Y.
SCANLAN, J. A., Evanston, Ill.
SCHUTT, W. G., Anchor Concrete Machinery Co., Adrian, Mich.
SCHAEFFER, J. F., National Lime Association, Chicago, Ill.
SCHIFRIN, I., Ideal Concrete Machinery Co., Cincinnati, Ohio.
SCHNEIDER, H. B., Heine Chimney Co., Chicago, Ill.
SCHNIEDWIND, J. C., Superior Supply Co., Chicago, Ill.
SCHODTLER, JOS. G., American Steel & Wire Co., Chicago, Ill.
SCHULMAN, H. M., Anchor Concrete Machinery Co., Chicago, Ill.
SCHURMANN, HOWARD, Independent Concrete Pipe Co., Indianapolis, Ind.
SHAFER, SAM, JR., Chain Belt Co., Milwaukee, Wis.
SHAFFER, W. H., JR., National Steel Fabric Co., Pittsburgh, Pa.
SHELHONY, R. H., Indianapolis, Ind.
SHENK, A. B., Federal Cement Tile Co., Chicago, Ill.
SIMPSON, LOUIS, 172 O'Connor St., Ottawa, Ont., Can.
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SMITH, ALLAN R., Portland Cement Association, Chicago, Ill.
SMITH, BLAINE S., Universal Portland Cement Co., Chicago, Ill.
SMITH, GEO. A., The Celite Co., Lompoc, Calif.
SMITH, H. E., Concrete Products, Chicago, Ill.
SMITH, LEE ROY, Indianapolis, Ind.
SMITH, H. A., G. A. Fuller Co., Chicago, Ill.
SMITH, ROBERT H., Castle Shannon, Pa.
SMOLEY, C. K., International Correspondence School, Scranton, Pa.
SMYERS, WM. H., Duquesne Slag Products Co., Pittsburgh, Pa.
SNOWBALL, J. R., Portland Cement Association, Columbus, Ohio.
SORENSON, JULIUS, Julius Sorenson & Sons, Racine, Wis.
SQUIRE, F. D. L., *Building Supply News*, Chicago, Ill.
STIELER, MARTIN, Heine Chimney Co., Chicago, Ill.
STEPANIAN, STEPHEN, The Arrow Sand & Gravel Co., Columbus, Ohio.
STEPHENS, A. W., Turner Construction Co., Chicago, Ill.
STEWARD, CHAS. A., Plano Concrete Works, Plano, Ill.
SUMMOND, F. P., Acme Cement Products Co., Cement City, Mich.
STEWART, G. M., C. N. Rys., Toronto, Ont., Can.
STILZ, FRED D., Irvington Coal Mine Co., Indianapolis, Ind.
STOKES, C. R., National Lime Association, Washington, D. C.
STONE, F. L., Universal Cement Co., Chicago, Ill.
STONE, P., *American Contractor*, Chicago, Ill.
STRAUB, F. J., Straub Cinder Block, New Kensington, Pa.
STREET, LOUIS J., Winnipeg, Man., Can.
STRIKER, EDMUND H., Baldwin, L. I., N. Y.
SULLIVAN, FRANK H., City Hall, Chicago, Ill.
SULLIVAN, ROBERT B., City of Chicago Bridge Dev., Chicago, Ill.
SWANSON, A. G., Omaha Concrete Stone Co., Omaha, Neb.
SWANSON, FRANK, Edmonton, Alta., Canada.
SWEETSER, E. O., Washington University, St. Louis, Mo.
SWEN, A. R., Iowa Railways Light Corp., Cedar Rapids, Iowa.
SWETT, L. W., Ransome Concrete Machinery Co., Chicago, Ill.
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TALBOT, K. H., Elgin, Ill.
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FEATHER STONE INSULATION CO., 911 Mateo St., Los Angeles, Calif. (Robt. Burhans, Jr., Pres.)

FEDERAL CEMENT TILE CO., 608 S. Dearborn St., Chicago, Ill. (Leland J. Wilhartz, Secy.)

FEDERAL CONCRETE CO., 667 Wyoming Ave., Buffalo, N. Y. (Walter E. Jones, Secy.)

FELDRAPPE, M. G., 16801 Williard Rd., Lakewood, Ohio. (A. A. Lane Construction Co.)

FERGUSON, JOHN A., 1419 N. Euclid Ave., Pittsburgh, Pa.

FERGUSON & CO., J. B., Hagerstown, Md. (J. B. Ferguson.)

*FERGUSON CO., JOHN W., United Bank Bldg., 152 Market St., Paterson, N. J. (John W. Ferguson.)

FERGUSON, LEWIS R., 342 Madison Ave., New York, N. Y. (International Cement Corporation.)

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*FERRO CONCRETE CONSTRUCTION CO., Third and Elm Sts., Cincinnati, Ohio. (W. P. Anderson.)

FERRO CONCRETE CONSTRUCTION CO., Third and Elm Sts., Cincinnati, Ohio. (H. D. Loring.))

FERRY CO., INC., JAMES, Virginia and Mediterranean Aves., Atlantic City, N. J. (James V. Ferry.)

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FISHER, EVAN THOMAS, 660 Market St., San Francisco, Calif.

FISHER, OTTO F., Vasagatan 38, Stockholm, Sweden. (Betongbyran.)

FISCHER-DEVORE CONSTRUCTION CO., 1025 Dixie Terminal Bldg., Cincinnati, Ohio. (Frank F. Fisher.)

*FISKE-CARTER CONSTRUCTION CO., 11 Foster St., Worcester, Mass. (Burton C. Fiske.)

FLEMING, GEO. S., 120 Madison Ave., Detroit, Mich. (Robert O. Derrick, Inc.)

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Fletcher, Matthews, 1660 W. Astor St., Indianapolis, Ind.
Fletcher-Thompson, Inc., P. O. Box 85, Bridgeport, Conn. (Edward A. Lambert.)
Flight, Oscar, Carpenter St., Bendigo Victoria, Australia.
Florida McCracken Concrete Pipe Co., Tampa, Fla. (J. R. Rankin.)
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Font, Manuel, Guayamo, Porto Rico.
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Friedmann, Carl A., 1112 Union Oil Bldg., 7th and Hope Sts., Los Angeles, Calif.
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Friske, A. W., Alois P. O., Wis.

FRITZ-RUMOR-COOKE CO., THE, 206 Interurban Bldg., Columbus, Ohio.
FROEHLING & ROBERTSON, Richmond, Va. (H. C. Froehling.)
FROST, CHAMBERLAIN & EDWARDS, Slater Bldg., Worcester, Mass.
FROST, HERMANN, c/o Jernbeton, Trondhjem, Norway.
FRUCHTBAUM, J., 726 Genesee Bldg., Buffalo, N. Y. (Truscon Steel Co.)
FRUIN-COLNON CONTRACTING CO., 502 Merchants-Laclede Bldg., St. Louis, Mo. (A. P. Greensfelder, Secy.)
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FURLONG, IRVING, 81 Appraiser Bldg., San Francisco, Calif. (Bureau of Standards.)
FUSEJIMA, SHINKURO, Engineering Dept., Chosen Gov. Railway Co., Ryusan, Chosen, Japan.
GABRIEL STEEL CO., 1150 Penobscot Bldg., Detroit, Mich. (W. F. Zabriskie.)
GAGE, ROBERT B., M. J. State Highway Dept., Box 106, Trenton, N. J.
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GALIEN CONCRETE TILE CO., Galien, Mich. (C. A. Roberts.)
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GARDNER, FRANC E., 3123 Bloomingdale Road, Chicago, Ill. (Gardner-Barada Chemical Co.)
GARDNER, FRANC J., 134 S. La Salle St., Chicago, Ill. (Atlas Portland Cement Co.)
GARTIES, GEORGE, 2400 Gilbert Ave., Cincinnati, Ohio.
GASTON, H. F., c/o Dunn Mfg. Co., Holland, Mich.
GAZDER, MAHAMED HASHIM, Lloyd Barrage & Canals Construction, Nawabshah, Sind, India.
GEDNEY CO., K. H., Hastings, Neb. (Kenneth H. Gedney.)
GEDNEY, RALPH, 1413 "F" St., N. E., Washington, D. C.
GENERAL BUILDING CO., INC., 524 Harrison Ave., Boston, Mass. (H. W. Marshall.)
GENERAL FIREPROOFING CO., Youngstown, Ohio. (W. B. Turner.)
GENERAL INDUSTRIAL ENGINEERING CO., 127 Prospect St., Passaic, N. J.
GERMUNDSSON, TH., 1421 Maple Ave., Evanston, Ill.
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GIBSON, JAMES E., Wood-Norton Apts., Germantown, Philadelphia, Pa.

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GILMAN, CHARLES, 50 Church St., New York City. (Massey Concrete Products Corp.)

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GINSBERG-PENN CO., 18 E. 41st St., New York, N. Y. (Frank I. Ginsberg.)

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*GLENS FALLS PORTLAND CEMENT CO., 205 Lower Warren St., Glens Falls, N. Y. (G. F. Boyle.)

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GODFREY, EDWARD, Monongahela Bank Bldg., Pittsburgh, Pa.

GODLEY, S. S. AND G. H., 716 Southern Railway Bldg., Cincinnati, Ohio. (George H. Godley.)

GOETZ, JOHN A., Mattoon, Ill.

GOGUEN, LEONARD E., 44 Fairfield St., Boston, Mass. (Structural Service Co.)

GOLDEN BAY CEMENT CO., LTD., THE, P. O. Box 446, Wellington, New Zealand.

GOLDIE MFG. CO., Trenton Ave. and P. R. R., Wilkinsburg, Pa. (Wm. Goldie, Jr.)

GOLDSMITH METAL LATH CO., THE, 3rd and Eggleston Ave., Cincinnati, Ohio. (Louis I. Zogoren.)

GONNERMAN, H. F., 1951 W. Madison St., Chicago, Ill.

GOTTLIEB, R. D., 2008 Post Dispatch, Houston, Texas.

GOTTSCHALK, L. F., Columbus, Neb.

GOULD, FRANK D., Fairmont, Minn.

GOULD, HARLEY J., c/o Ferro-Concrete Constr. Co., Cincinnati, Ohio.

GOULD CONTRACTING CO., 1214 Ind. Life Bldg., Nashville, Tenn. (C. B. Wilson.)

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GRAF, OTTO, Stuttgart-Villastrasse 12, Germany.

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GRANITE ROCK CO., Watsonville, Calif. (A. J. Wilson.)

GRASSELLI CHEMICAL CO., THE, 1300 Guardian Bldg., Cleveland, Ohio. (R. F. Remler.)

GRAVES, FRANK W., 326 Beaver Hall Hill, Montreal, Canada. (J. S. Archibald.)

GRAY CONCRETE CO., Thomasville, N. C. (F. B. Gray.)

GRAY CONTRUCTION CO., LTD., J. V., 541 Queen St., E., Toronto 2, Ont. (R. J. Fuller.)

GRAY, HOWARD ALLISON, 862 Park Square Bldg., Boston, Mass. (Morton C. Tuttle Co.)

GREEN CO., SAMUEL M., 496 Bridge St., Springfield, Mass. (Samuel M. Green.)

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GREENMAN, RUSSELL S., State Engineer's Dept., Albany, N. Y.

GREINER, J. E., Lexington Bldg., Baltimore, Md.

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GULF CONCRETE PIPE CO., P. O. Box 1765, Houston, Texas. (N. A. Eppes.)
GURTLER, WM. A., 506 Harvester Bldg., Chicago, Ill. (James O. Heyworth, Inc.)
GURUSWAMI, S., P. O. Bhira (via) Nagothna, Kolaba District, Bombay, India.
HADDON, HUGH, Millville, N. J.
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HALL & STEVENSON, 409 White Bldg., Seattle, Wash. (J. H. Stevenson.)
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HAMMILL, HAROLD B., 42 Portsmouth Road, Piedmont, Oakland, Calif.
HAMMITT, ANDREW B., 312 Broad Street Bank Bldg., Trenton, N. J. (Par-Lock Appliers of N. J.)
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HANNA, H. B., R. R. No. 1, Prescott, Ont.
HANNAFORD, H. ELDRIDGE, 1024 Dixie-Terminal Bldg., Cincinnati, Ohio. (S. Hannaford & Sons.)
HANSARD, ORREN H., Tenn. Dept. of Highways and Public Works, Nashville, Tenn.
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HARDY, RICHARD, 1011 James Bldg., Chattanooga, Tenn. (Dixie Portland Cement Co.)
HARESNAPE, V., 800 Corporation Bldg., Los Angeles, Calif. (Riverside Portland Cement Co.)

HARGEN, STANLEY, 133 Rutland Road, Brooklyn, N. Y. (Standard Oil Co.)
HARIG CONSTRUCTION CO., ROBT., 2174 Western Ave., Cincinnati, Ohio. (Ben Harig.)
HARMAN, H. H., Greenville, Pa.
HARMS, H. J., Rotterdam, Holland. (Continental Petroleum Co.)
HARRINGTON, JOHN LYLE, 1012 Baltimore Ave., Kansas City, Mo. (Harrington, Howard & Ash.)
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HARRISBURG BUILDING BLOCK CO., Cameron and Reilly Sts., Harrisburg, Pa. (J. Edwin Rutter.)
HARRISON CONSTRUCTION CO., J. S., 2012 Amicable Bldg., Waco, Texas. (C. H. Harrison.)
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HAYES ENGINEERING CORP., 49 Taku Rd., Tientsin, China. (J. J. Davison.)
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*HELDERBERG CEMENT CO., Albany, N. Y. (Charles R. Parks.)
HELLER, MARTIN, Granite City, Ill.
HELLER-MURRAY CO., 222 W. Rayen St., Youngstown, Ohio. (A. H. Heller.)
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HENDRICK, JEAN, 125 Quai de Valmy Paris (Xeme), France. (Poliet & Chausson.)
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HERSEY CO., LTD., MILTON, 84 St. Antoine St., Montreal, Canada. (Walter C. Adams.)
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HILKER SUPPLY CO., 16th and State Sts., Granite City, Ill. (E. W. Hilker.)
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HINDMAN, W. S., 172 E. North Broadway, Columbus, Ohio.
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HOEFFER & CO., Chamber of Commerce Bldg., Chicago, Ill. (Alexander C. Warren.)
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HOFFMAN & CO., F., 901 Wood St., Wilkinsburg, Pa. (Wm. F. Hoffman.)
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HOGAN, JAMES, 169 Sassoon Dask, Calaha, Bombay, India.
HOLABIRD & ROCHE, 104 S. Michigan Ave., Chicago, Ill. (E. A. Renwick.)
HOLLISTER, S. C., 311 Elm St., Swarthmore, Pa.
HOLLOW BUILDING TILE ASSOCIATION, 1409 Conway Bldg., Chicago, Ill. Frank J. Huse.)
HOLLYWOOD BUILDING BLOCK CO., North Plymouth, Pa. (Henry C. Parker, Sec'y and Treas.)
HOLM, W. M., c/o Chief Engr., A. T. & P. Ry., Amarillo, Texas.
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HOLMES, FRANCIS, 248 Lambton Quay, Wellington, New Zealand.
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HOLTER, A., Brevik, Norway.
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HOPPER, GEORGE B., Pres., Van Guilder System Concrete Bldg., Inc., 15 E. 40th St., New York, N. Y.
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HORN, H. M., 17 Battery Place, New York City.
HORNER, WESLEY W., 300 City Hall, St. Louis, Mo.
HORST CO., HENRY W., Horst Bldg., Rock Island, Ill. (A. E. Horst.)
HORTH, A. J., JR., 502 Terminal Bldg., Youngstown, Ohio.
HOUGHTON, JAMES R., 276 Ridge Rd., Lyndhurst, N. J.
HOUK, HOWARD H., U. S. Bureau of Public Roads, Montgomery, Ala.
HOUSEMAN ROOFING CO., INC., 1521 Pierre Ave., Shreveport, La. (G. A. Houseman, Pres.)
HOWE, C. D., The Whelan Bldg., Port Arthur, Ont., Canada.
HOWE, H. N., 76 Porter Bldg., Memphis, Tenn.
HOWES & MCGINTY, INC., 64 Whitman St., New Bedford, Mass. (John J. McGinty.)
HOYER-ELLEFSEN, A/s, P. O. Box 39, Oslo, Norway. (August Gundersen.)
HOYT, W. A., Altoona, Pa.
HU, K. P., cor. Rue Verdun and Rue Takou, Tientsin, China. (Chee Hsin Cement Co., Ltd.)
HU, KUANG TAO, 2 S. Clinton Ave., Trenton, N. J.

HUBBARD, FRED, 707 Wick Bldg., Youngstown, Ohio.
HUBBELL-HARTGERING & ROTH, 2640 Buhl Bldg., Detroit, Mich. (Clarence W. Hubbell.)
HUBNER, DR. FRIEDERICH, Portland Cement Works, Balingen, Germany.
HUDSON, JAMES A., Memphis, Tenn. (Dist. Engr., Portland Cement Assn.)
HUDSON, R. J. H., Public Works Dept., Ranchi, India.
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HUGHES, L. E., 315 W. 98th St., New York, N. Y.
HUGHES, R. G., 152 Market St., Paterson, N. J. (John W. Ferguson Co.)
HUKMANI, S. D., Assistant Engineer, Lloyd Barrage, Rohri, Sind, India.
HUMBOLDT GRAVEL AND TILE CO., Humboldt, Iowa.
HUME, ALBERT S., Cangallo 465, Buenos Aires, Argentine.
HUME PIPE CO. (SOUTH AFRICA), LTD., National Bank Bldgs., Simmonds St., Johannesburg, South Africa. (Walter Wolstenholme.)
HUME, WALTER REGINALD, Reliance House, 301 Flinders Lane, Melbourne, Victoria, Australia. (Hume Steel, Ltd.)
HUMPHREY, D. S., Euclid Beach Park, Cleveland, Ohio. (The Humphrey Co.)
HUMPHREY, RICHARD L., Real Estate Trust Bldg., Philadelphia, Pa.
HUNDHAUSEN, WM., 421 Barker Ave., Peoria, Ill.
HUNT & CO., ROBERT W., 53 Park Place, New York City. (J. F. Davis.)
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HUNTER, HARRY B., Roof U. & P. Bldg., Memphis, Tenn.
HURLBURT, R. W., 100 Jarvis St., Toronto, Ont.
HURLEY, JOHN J., 121 Walnut St., Sommerville, Mass.
*HURON PORTLAND CEMENT CO., 1525 Ford Bldg., Detroit, Mich. (John W. Boardman.)
HUSER, DR. ING. ALFRED, Obercassel-Siegkreis, Germany.
HUSTAD CO., THE, 126 S. 9th St., Minneapolis, Minn. (A. P. Hustad, Pres.)
HUTCHINSON, G. W., Commercial Bank Bldg., Raleigh, N. C.
HUTTER CONSTR. CO., 128 Western Ave., Fond du Lac, Wis. (Geo. F. Hutter.)
HYDE, STANLEY T., 212 9th St., Bremerton, Wash.
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HYNES, W. J., LTD., 859 Dupont St., Toronto, Can.
IDEAL CEMENT STONE CO., 31st and Spaulding St., Omaha, Neb. (A. V. Johnson.)
IDEAL CEMENT WORKS, 1013 W. Front St., Grand Island, Neb. (Alfred L. Mader.)
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IMMEL CONSTRUCTION CO., 98 N. Main St., Fond du Lac, Wic. (Harry W. Mabie, Jr.)

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*INDEPENDENT CONCRETE PIPE CO., 201 N. West St., Indianapolis, Ind. (Howard Schurmann.)

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*INSLEY MFG. CO., P. O. Box 167, Indianapolis, Ind. (Alvin C. Rasmussen.)

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IRWIN & LEIGHTON, 126 N. 12th St., Philadelphia, Pa. (E. M. Campbell.)

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JOHNSON, VIRGIL L., 19th St., above Chestnut St., Philadelphia, Pa.

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KALMAN FLOOR CO., 410 N. Michigan Ave., Chicago, Ill. (C. E. Cooke.)

*KALMAN STEEL CO., 410 Michigan Ave., Chicago, Ill. (William S. Thomson.)

*KALMAN STEEL CO., 410 N. Michigan Ave., Chicago, Ill. (A. P. Clark.)

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Klinger, W. A., Warnock Bldg., Sioux City, Iowa.

Klock, Morgan B., 343 Alexander St., Rochester, N. Y.

Kneeshaw, F. P., 64 Gett St., Sydney, N. S. W., Australia. (Kandos Cement Co., Ltd.)

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Kober, Wm., Co., c/o Adensite Co., Inc., 116 W. 39th St., New York City.

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*Koehring Company, 4940 N. 8th St., Philadelphia, Pa. (P. Koehring.)

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Koleser, Andrew, Phillipsburg, N. J.

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Krause, Mark C., 120 W. 4th St., Williamsport, Pa.

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LAKEWOOD ENGINEERING CO., Cleveland, Ohio. (Lion Gardner, Vice-Pres.)

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*LEHIGH PORTLAND CEMENT Co., Allentown, Pa. (H. M. Trexler, Pres.)

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*LEHIGH PORTLAND CEMENT Co., Young Bldg., Allentown, Pa.

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MANITOWOC PORTLAND CEMENT CO., Manitowoc, Wis. (P. G. Dawson.)

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MARISCAL, FREDERICO E., 9a Colima 292, Mexico City, Mexico.
MARKLAND, M. B., Guarantee Trust Bldg., Atlantic City, N. J.
MARLBORO CEMENT CO., Edmonton, Alberta, Can. (A. W. G. Clark.)
MARQUES, CHARLES, St. Margarets, Ware, Hertfordshire, England.
MARQUES, CHAS. A., JR., c/o Messrs. Concrete Utilities, Ltd., Herts, England.
MARQUESS, C. H., c/o Armour & Co., Union Stock Yards, Chicago, Ill.
MARQUETTE CEMENT MFG. CO., LaSalle, Ind. (C. M. Butler.)
*MARQUETTE CEMENT MFG. CO., Marquette Bldg., Chicago, Ill. (R. B. Dickinson.)
MARSCH, LOUIS, Morrisonville, Ill.
MARSH, EARL G., 922 Maryland Ave., N. E., Washington, D. C.
MARSH-MURDOCK CO., THE, Melish and Stanton Aves., Cincinnati, Ohio. (George J. Marsh.)
MARSHALL, JOHN, 49 Locket Road, Wealdstone, Harrow, Middx., England.
MARSHALL, JOS. M., JR., 1319 Hurt Bldg., Atlanta, Ga. (Dist. Engr., Portland Cement Assn.)
MARSHALL, THOS. W., 1341 Connecticut Ave., Washington, D. C.
MARSON, JOHN E., Aurora, Ill. (Barber-Greene Co.)
MARTIN, EDGAR, Chicago Beach Hotel, Chicago, Ill.
MARTIN, EVAN S., 16 Saulter St., Toronto, Ont. (James A. Wickett, Ltd.)
MARTIN, FRANK J., 3014 Broadway, San Antonio, Texas. (c/o Jefferson Constr. Co.)
MARTIN, G. G., R. D. 1, Meadville, Pa. (Bessemer and Lake Erie Railroad Co.)
MASON, FRANK H., 2 Greylock Rd., Waterville, Me. (Central Maine Power Co.)
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MASTERS, F. M., Const. Engr., Calder Bldg., Harrisburg, Pa.
MATTHEWS, HOMER M., 1112 Kirby Bldg., Dallas, Texas.
MAURO, FRANCESCO, 1405 Beech St., Birmingham, Ala.
MAURY, GEO. R., Mineral Wells, Texas. (Mineral Wells Crushed Stone Co.)
MAYERS, H. WINFIELD, No. 8 Wilson Ave., Watertown, Mass.
MAYNARD, ARTHUR J., Mass. State Farm, State Farm, Mass.

MAYNICKE & FRANKE, 25 Madison Square, North, New York City, N. Y.
MEAD, C. A., 165 Wildwood Ave., Upper Montclair, N. J.
MEADE, SUYDAM CO., 342 6th Ave., Newark, N. J. (F. J. Meade.)
MEADE, P. F., Denver, Colo. (Dist. Engr., Portland Cement Assn.)
MEDFORD CONCRETE CO., Medford, N. J. (Harry L. King, Jr., Pres.)
MEHTA, B. M., Mody Bldg., No. 20, Ghatkopar, Bombay, India.
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MELTZER, JOSEPH, 590 Fort Washington Ave., New York, N. Y.
MENEFEE, F. N., 104 West Eng. Bldg., Ann Arbor, Mich.
MERCHANT, ARCHIE W., 728 Hospital Trust Bldg., Providence, R. I.
MERLO, MERLO & RAY, LTD., Ford, Ont., Canada. (Louis Alvin Merlo.)
MERRIMAN, THADDEUS, 2224 Municipal Bldg., New York, N. Y.
MESSEY, LAUREL, Commerce Bldg., Ash and George Sts., Sydney, Australia.
METCALF & EDDY, 14 Beacon St., Boston, Mass. (Frank A. Marston.)
METZGER, F. KINSEY, 6321 Delaware St., Chevy Chase, Md.
METZGER-RICHARDSON COMPANY, 503 May Bldg., 529 Liberty Ave., Pittsburgh, Pa. (F. L. Metzger.)
MEYER, C. LOUIS, 608 Omaha National Bank Bldg., Omaha, Neb. (Concrete Engrg. Co.)
MEYER, MORRISON & CO., 39 Cortlandt St., New York City. (B. A. Meyer.)
MEYER, WM. H., 1468 Maryland Ave., Milwaukee, Wis.
MICHAELIS, FRIEDRICH, Dusseldorf (Rheiland), Rathausufer No. 19, Germany.
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MIESENHELDER, P. D., 640 Middle Drive, Woodruff Pl., Indianapolis, Ind. (Indiana State Highway Commission.)
MIKI, MIYOKICHI, 439 Kashiwagi, Yodobashimachi, Tokyo, Japan.
MILBURN LIME & CEMENT CO., LTD., 59 Crawford St., Dunedin, New Zealand. (J. H. Stewart, Gen. Mgr.)
MILLER & CO., E. S., 429 Broadway St., Milwaukee, Wis. (R. W. Stambaugh.)
MILLER & SONS' CO., H., 2565 5th Ave., Pittsburgh, Pa. (A. G. Miller.)
MILLER, G. R., S. M. Damon Bldg., Honolulu, Hawaii.
MILLER, L. C., 610 Merchants Bank Bldg., Indianapolis, Ind. (Dist. Engr., Portland Cement Assn.)
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*MISSOURI PORTLAND CEMENT CO., Post Dispatch Bldg., St. Louis, Mo. (H. L. Block, Pres.)

MITCHELL, JAMES, 999 Bergen Ave., Jersey City, N. J.

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MIZUKAMI, S., 16 Ukyo-Machi, Yotsuya-Ku, Tokyo, Japan.

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MOHLER, JOHN D., 2740 Duncan St., St. Joseph, Mo.

MOHR, H. A., Apt. 56, 2325 University Ave., New York, N. Y. (Dist. Mgr. Raymond Concrete Pile Co.)

MOLE, HARRY H., City Engineer, Kearney, Neb.

MOLINE CAST STONE CO., 110 18th St., Moline, Ill. (Richard Bjoindakl.)

MOLLENKOF, J. P., c/o John H. McClatchy, Erdenheim, Montgomery Co., Pa.

MONARCH ENGINEERING CO., Chamber of Commerce Bldg., Buffalo, N. Y. (H. R. Wait, Pres.)

MONKS & JOHNSON, 99 Chauncey St., Boston, Mass. (John J. Harty.)

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MORRIS, L. E., Valley Center, Kan.

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MUELLER, J. W., Palladium Bldg., Richmond, Ind.
MUIRHEAD CONSTRUCTION CO., WM., Durham, N. C. (Wm. Muirhead.)
MULHAUSEN, LOUIS, 10 S. 18th St., Philadelphia, Pa.
MUNICIPAL RESEARCH BUREAU OF CLEVELAND, 403 Electric Bldg., Cleveland, Ohio. (H. P. Cummings.)
MUNTZ, E. P., 403 Lehigh Valley Terminal, Buffalo, N. Y.
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NAITO, TACHU, University of Waseda, Tokyo, Japan. (Engineering College.)
NAKAGAWA, H., Tokio Kaijo Bldg., Tokio, Japan. (Asano Cement Co.)
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NASU, AKIYA, Kawasaki Works, Nakashibuya, No. 715, Tokyo, Japan.
NATIONAL CONCRETE CONSTRUCTION CO., 54 Bd. of Trade, Louisville, Ky. (J. B. Ohligschlager.)
NATIONAL CRUSHED STONE ASSN., 751 Earle Bldg., Washington, D. C. (A. T. Goldbeck.)
NATIONAL FIREPROOFING CO., Flatiron Bldg., New York City. (P. Bevier.)
NATIONAL LIME ASSOCIATION, 918 G St., N. W., Washington, D. C. (W. A. Freret.)
NATIONAL LUMBER MFGS. ASSN., 402 Transportation Bldg., Washington, D. C. (D. F. Holtman.)
NATIONAL STONE TILE CORP., 15 E. 40th St., New York, N. Y. (C. C. H. Thomas.)
NATIONAL TESTING LABORATORIES, LTD., THE, 223 James St., Winnipeg, Man., Canada. (L. J. Street.)
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*NAZARETH PORTLAND CEMENT CO., Nazareth, Pa.
NEAL GRAVEL CO., Mattoon, Ill. (E. Guy Sutton.)
NEFF & FRY CO., THE, Camden, Ohio. (C. Rodney Neff.)
NELSON-ENBLOM CO., 917 Plymouth Bldg., Minneapolis, Minn. (Albert Enblom, Sec'y-Treas.)
NEPENNA BUILDING MATERIALS CO., P. O. Box 427, Kingston, Pa. (F. L. Schott.)
NEUMAN, DR. ERWIN, Prof., Technische Hochschule, Stuttgart, Germany.
*NEWAYGO PORTLAND CEMENT CO., Newaygo, Mich. (J. D. John.)
NEW EGYPTIAN PORTLAND CEMENT CO., 408 W. Fort St., Detroit, Mich. (E. R. Sullivan.)
NEWELL, NEAL, Iona, N. J.

NEW JERSEY ZINC CO., Palmerton, Pa. (Technical Library.)
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NICHOLS, CHARLES ELIOT, 147 Milk St., Boston, Mass. (Stone & Webster, Inc.)
NICHOLSON, JR., JOHN, 2735 Prospect Ave., Cleveland, Ohio.
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NOICE, BLAINE, 1326 Washington Bldg., Los Angeles, Calif.
NOONAN, W. H., 334 Roy Bldg., Halifax, Nova Scotia.
NORRIS, J. F., 107 Norris St., Rochester, N. Y.
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NORTHERN CONSTRUCTION CO., LTD., Vancouver Block, Vancouver, B. C., Canada. (Wm. Smaill, Chief Engr.)
NORTH JERSEY DIST. WATER SP. COMM., 24 Commerce St., Newark, N. J. (Arthur H. Pratt.)
NORTHWESTERN STATES PORTLAND CEMENT CO., Mason City, Iowa. (G. C. Blackmore.)
NOVELLA, GUSTAVO, Avenida del Hipodromo, Gautemala, Guatemala, C. A.
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OHIO RIVER SAND CO., 129 River Road, Louisville, Ky. (J. W. Settle, Sec'y.)
OKLAHOMA PORTLAND CEMENT CO., Ada, Okla. (O. A. Bayless.)
OKUBO, TOSHIYUKI (Truscon Steel Co. of Japan), Yurakucho Kojimachi, Tokyo, Japan.
OLSON, OLE K., 722 Pendido St., New Orleans, La.
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OTTO, EDGAR D., Downer Grove, Ill.

OUTZEN, ANDREW N., 415 Clifford, Detroit, Mich. (Supt. River Rouge Co.)

OWEN, THOS. W., City Engineer, Port Angeles, Wash.

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*PACIFIC PORTLAND CEMENT CO., 827 Pacific Bldg., San Francisco, Calif.

PADILLA, GUSTAVO E., Once de Agosto St. No. 26, Mayaguez, Porto Rico.

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PEASE, B. S., 208 S. LaSalle St., Chicago, Ill. (Am. Steel & Wire Co.)

PEASE, F. A., 804 Marshall Bldg., Cleveland, Ohio.

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PEELESS ARTIFICIAL STONE, LTD., 514 Coxwell Ave., Toronto, Ont., Canada. (F. C. Bee.)

PEERLESS CONCRETE PRODUCTS CO., 4956 8th Ave., S., Seattle, Wash.

*PEERLESS PORTLAND CEMENT CO., Union City, Mich. (Wm. M. Hatch.)

*PENINSULAR PORTLAND CEMENT CO., Cement City, Mich.

*PENN-ALLEN CEMENT CO., Nazareth, Pa.

*PENN-ALLEN CEMENT CO., Widener Bldg., Nazareth, Pa. (W. E. Eidel.)

PENN BUILDING BLOCK CO., INC., 807 Chestnut St., Philadelphia, Pa. (Raymond M. Weeks.)

*PENNSYLVANIA CEMENT CO., 131 E. 16th St., New York City. (William Beach.)

*PENNSYLVANIA CEMENT CO., 131 E. 16th St., New York, N. Y. (W. N. Beach, Pres.)

PENNSYLVANIA STATE HIGHWAY DEPT., Harrisburg, Pa. (W. H. Connell, Asst. State Highway Comm.)

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PIERSON CONCRETE PRODUCTS CO., 89 Dodd St., East Orange, N. J. (James T. Pierson, Vice-Pres.)

PIGOTT, JOSEPH M., Pigott-Healy Construction Co., Hamilton, Ont., Can.

PINGREY & CO., R. E., 134 S. LaSalle St., Chicago, Ill. (R. E. Pingrey.)
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PLASTIC PRODUCTS CO., 123-125 Reservoir Ave., Milwaukee, Wis. (Ralph W. Albrecht, Pres.)
PLATOFF & BUSH, 122 W. Liberty St., Louisville, Ky. (Carl Schneider.)
PLUMER, H. E., 22 Ellicott Square, Buffalo, N. Y.
POLARIS CONCRETE PROD. CO., Box 86, W. Duluth, Minn. (E. H. Dresser, Pres.)
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PORISS CO., THE S. C., 198 N. Quaker Lane, West Hartford, Conn. (S. C. Poriss.)
PORTER, J. M., Easton, Pa.
PORTER, LEO A., Arlington Hotel, Lehighton, Pa.
POST & MCCORD, 101 Park Ave., New York, N. Y. (Andrew J. Post, Pres.)
POWELL, F. H., 605 H. W. Hellman Bldg., Los Angeles, Calif.
POWER, S. M., School of Mines, Bendigo, Victoria, Australia.
POWERS, JOHN M., 609 W. 3rd St., Sterling, Ill.
POWERS & SON, EUGENE S., 1520 W. Locust St., Philadelphia, Pa. (E. S. Powers.)
POWERS KENNEDY CONTRACTING CORP., 149 Broadway, New York City. George C. Bingham.)
POZZO, ALBERTO, Corso Re Umberto 63, Torino, Italy.
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PRATT, H. B., Antrim, N. H.
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PRINCE CONCRETE CO., 212 North 38th St., Camden, N. J. (G. R. Prince & Co.)
PRITCHETT, FRANK S., North Carolina Amiesite & Stone Co., First Bank & Trust Bldg., Hendersonville, N. C.
PROBST, EMIL, Karlsruhe i B., Germany. (Technische Hochschull.)
PROPS OF HAY'S WHARF, LTD., Tooley St., London, S. E. I. (H. J. M. Skidmore.)
PUCCI, ANGELO, 332 First St., Rochester, N. Y.
PUNJAB PORTLAND CEMENT, LTD., Wah. N. W. Railway, Attock District, Punjab, India. (T. Campbell Gray, Mgr.)
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QUEBEC DEPT. OF ROADS, Parliament Bldgs., Quebec, P. Q. (J. L. Baulanger.)

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REED CO., WILLIAM T., 200 Devonshire St., Boston, Mass. (William T. Reed.)

REGAN, P. E., Box 733, Route 3, Jacksonville, Fla. (Arnold Stone Brick & Tile Co.)

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RHODE, HOWARD, 14 S. Munn Ave., East Orange, N. J. (International Cement Corp.)

RIB-STONE CONCRETE CORP., LeRoy, N. Y. (Geo. E. Priest.)

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RICH, MELVIN S., 1410 H St., N. W., Washington, D. C.

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VAUCLAIN, JACQUES L., Buck Lane, Bryn Mawr, Pa.

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VORTEX MFG. CO., 1978 W. 77th St., Cleveland, Ohio. (Chas. H. Parkin.)

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VOSS, WALTER C., 22 Garrison Road, Brookline, Mass.

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VULCANITE PORTLAND CEMENT CO., 350 Madison Ave., New York City. (Albert Moyer.)

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WALTER, ERNEST, 263 Midland Ave., Montclair, N. J. (Porete Mfg. Co.)

WALTER, OTTO, Perryville, Ind.

WALTERS, EDWARD, Supt. Arnold Stone, Brick & Tile Co., P. O. Box 3039, Jacksonville, Fla.

WANG, YUAN KONG, Amer. Bridge Co., Elmira, N. Y.

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Warner Co., Chas., Franklin Bldg., Philadelphia, Pa. (F. S. Lee.)

Warren Co., F. V., 1913 Arch St., Philadelphia, Pa. (F. V. Warren.)

Warren, George E., 33 W. Grand Ave., Chicago, Ill. (Portland Cement Assn.)

Warren, Samuel, 25 Broadway, New York City. (Atlas Portland Cement Co.)

Wassef, K. Zaki, 14 Sh. Boutros, Heliopolis, Egypt.

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Wausau Concrete Co., 211 Jefferson St., Wausau, Wis. (C. W. Parsons.)

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Webb-Clark Co., 422 W. Liberty St., Louisville, Ky.

Webber, Chas. L., 4258 Raymond Ave., Los Angeles, Calif.

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Weiller, Chas. A., 617 Lexington Bldg., Baltimore, Md. (Concrete Steel Co.)

Weinberger, A. J., Beach, N. D.

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Wells, P. A., 914 Monadnock Block, Chicago, Ill. (Wells Bros. Constr. Co.)

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Western Precipitation Co., 1016 W. Ninth St., Los Angeles, Calif.

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WHITNEY BROS. CO., 909 Alworth Bldg., Duluth, Minn. (Gwin A. Whitney.)

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WIDMER, ARTHUR J., 941 Century Bldg., St. Louis, Mo. (Widmer Engineering Co.)

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WILLINGHAM, P. B., State Road Dept., Vero, Florida.
WILLSON, S. H., 404 Candler Bldg., Atlanta, Ga. (Kalman Floor Co.)
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WILSON, ARTHUR J., 5 Albion Place, Port Richmond, N. Y.
WILSON, FRANCIS F., 3 Wilson Lane, Rockville, Centre, N. Y.
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WILSON, JOHN, City Engr., Duluth, Minn.
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WITT, J. C., 510 W. 70th St., Chicago, Ill.

WITT, W. H., 204 Seaboard Bldg., Seattle, Wash.

WITTE, HERMAN CALVERT, 4626 Eastern Ave., Seattle, Wash.

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WRIGHT, EVERETT G., c/o Albert Kahn, Marquette Bldg., Detroit, Mich.

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WRIGHT & KREMERS, INC., Box 435, Niagara Falls, N. Y. (Ernest Kremers.)

WU, POND S., Supervisor of Works, Govt. Salt Administration, Peking, China.

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ZASS, W. W., c/o State Highway Commission.

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